AECOM

Submitted to:
US EPA Region 8
Denver, CO

Submitted by:
AECOM
Denver, CO
60306687
October 30, 2013

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Preliminary Design Report
St. Louis Tunnel Hydraulic Control Measures

Rico-Argentine Mine Site – Rico Tunnels
Operable Unit OU01
Rico, Colorado

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Operable Unit OU01
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October 30, 2013

VIA EMAIL AND HAND DELIVERY

Mr. Steven Way
On-Scene Coordinator
Emergency Response Program (8EPR-SA)
US EPA Region 8
1595 Wynkoop Street
Denver, CO 80202-1129

Subject: Preliminary Design Report, St. Louis Tunnel Hydraulic Control Measures
Rico Argentine Mine Site – Rico Tunnels Operable Unit OU01 Rico, Colorado

Dear Mr. Way,

A digital file in PDF format of the Preliminary Design Report, St. Louis Tunnel Hydraulic Control Measures, Rico Argentine Mine Site – Rico Tunnels Operable Unit OU01 Rico, Colorado, dated October 30, 2013, is being submitted to you today via email. Three (3) hard copies of the report will also be hand-delivered to your office no later than close of business November 1.

Atlantic Richfield Company (AR) is submitting this report responsive to requirements in Task D of the Removal Action Work Plan accompanying the Unilateral Administrative Order for Removal Action, Rico-Argentine Site, Dolores County, Colorado, U.S. EPA Region 8, Docket No. CERCLA-08-2011-0005.

If you have any questions, please feel free to contact me at (951) 265-4277.

Sincerely,

Anthony R. Brown

Anthony R. Brown
Project Manager
Atlantic Richfield Company

cc: Ronald Halsey, Atlantic Richfield Company (via e-mail)
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List of Acronyms

ac-ft	acre feet
cf	cubic feet
cy	cubic yard
EPA	Environmental Protection Agency
ft	feet
HSSE	health, safety, security and environmental protection
ID	inside diameter
NW	northwest
PDR	Preliminary Design Report
RCP	reinforced concrete pipe
SE	southeast
sf	square feet
SLT	St. Louis Tunnel
SSR-A	South Stack Repository - Alternative A
TM	Technical Memorandum
UAO	Unilateral Administrative Order
USFS	United States Forest Service

1.0 Introduction

1.1 Rationale

The St. Louis Adit Hydraulic Control Measures Project addresses specific requirements of the United States Environmental Protection Agency (USEPA) Unilateral Administration Order (UAO) (EPA, 2011a and Remedial Action Work Plan (EPA, 2011b), specifically Subtask D2 of Task D, “Preliminary Design of Hydraulic Controls of the Adit Discharge”. The location of the study area for the adit hydraulic control measures project is shown on Figure 1.1.

1.2 Objectives

The objectives of the St. Louis Tunnel (SLT) Adit Hydraulic Control Measures Project are to: 1) gather and convey essentially all of the tunnel discharge (to the extent practicable) to the selected water treatment system in a controlled manner; and 2) mitigate the release of settled solids and debris that may have accumulated in the SLT behind the blockage in the collapsed adit area.

As part of a final remedy for treatment of mine water discharging from the SLT, the open, collapsed portion of the tunnel at the west end of the excavation into the face of CHC hill (i.e., the terrain trap) will be reconstructed to remove colluvial debris and timber supports from the end of the tunnel, allow accurate measurement of mine water discharge, and facilitate conveyance of that mine water to the final treatment system.

An evaluation to assess the potential for attenuation of seasonally or climatically higher tunnel flows by storage within the SLT and interconnected mine workings concluded that such attenuation storage was not an attainable objective for this element of the overall site remedy. On an average annual basis (let alone a recurrence-year basis) utilization of the limited storage available would result in rapid and very substantial increase in the head within the SLT. The implications of such in-mine storage and the associated high heads include: 1) uncontrolled seepage discharge to the face of CHC Hill and slopes in the Silver Creek drainage; 2) destabilization of colluvium and existing landslide deposits on CHC Hill and slopes in the Silver Creek drainage; 3) increased hydraulic stress on the debris plug; and 4) wetting and possibly increased dissolution of metals from the tunnel/mine workings walls over what would occur otherwise.

1.3 Scope and organization

A final set of alternatives to achieve the objectives in Section 1.2 was adopted following consideration and preliminary evaluation of a wide array of potential concepts. This Preliminary Design Report (PDR) presents: 1) the technical characterization and hydraulic and geotechnical considerations of six short-listed alternatives; 2) a comparative evaluation among the alternatives resulting in a recommended alternative and back-up alternative to carry forward; and 3) preliminary (i.e., 30-percent) design of the recommended and back-up alternatives.

Section 1.0 presents the rationale, objectives, and scope and organization of the study, and provides a brief summary of the six action alternatives. Section 2.0 briefly characterizes the site setting and presents key background information regarding site conditions that together are the basis for developing the appropriate range of alternatives to be evaluated and for preparing preliminary designs for the recommended and back-up alternatives. Section 3.0 lists key design criteria to be met by the alternatives. Each of the alternatives is described in Section 4.0 in terms of their components and function, construction, and monitoring. This includes exhibits illustrating the key elements of the alternatives in plan, profile and where appropriate with selected cross sections. Section 5.0 presents the hydraulic conditions, opportunities and constraints applicable to the design, construction and operation of the alternatives. Geotechnical analyses relevant to the performance of the alternatives that incorporate continued use of the existing debris plug, and of the stability of existing slopes in the terrain trap and adjacent ground, are presented in Section 6.0. A qualitative risk assessment of the six action alternatives is presented in Section 7.0. A comparative evaluation of the alternatives and recommendation of two alternatives (a recommended and back-up alternative) to carry forward to preliminary design are presented in Section 8.0. Section 9.0 describes the preliminary design of the recommended and back-up alternatives.

1.4 Description of alternatives

Table 1.1 presents a summary of the alternatives identified and characterized to address the objectives presented in Section 1.2 above. As noted in Section 1.3, the key components of each alternative are discussed in Section 4.0.

2.0 Setting/Background

The Adit and Portal Investigation Report – 2013 Update (Atlantic Richfield Company, 2013) documents extensive geologic, geotechnical and geophysical field exploration and geotechnical laboratory testing performed at the collapsed adit area of the St. Louis Tunnel over the past three years. That report also presents a detailed characterization of the site in Section 5.0 based on the field and laboratory investigations. A brief summary of the history of the SLT in this area and of the most relevant geologic/geotechnical conditions present follow.

This evaluation of alternatives and preliminary design of recommended alternatives for this part of the overall remedy are focused on the portion of the SLT that was originally driven through approximately 330 feet of colluvium at the base of CHC Hill, and then into bedrock of the Hermosa Formation to just beyond the reach that was reportedly lagged (i.e., approximately 35 feet into the rock from the contact with colluvium). Based on archival tunnel geologic mapping and historic photographs, it is inferred that the tunnel is nominally seven (7) feet high and nine (9) feet wide.

Available aerial photographs show that a major excavation was made over the downgradient reach of the SLT sometime between August 1950 and October 1952. The resultant steeply sloping U-shaped excavation is now referred to as the “terrain trap”. The slopes in this area are excessively steep and judged at best metastable to unstable at their angle of repose. Cobble- and boulder-size rocks roll and tumble down these slopes, especially following rainfall and snowmelt events. Finer (sand and gravel fraction) colluvium also continues to be transported down slope by gravity, accumulating at the toes of the slopes.

Review of subsequent aerial photography is interpreted to show that the remaining ground over this reach of the tunnel remained relatively undisturbed until at least August 1989, except that raveling of the slopes removed the benches visible shortly after the initial excavation. Sometime later, inferred by review of available aerial photography as before September 1998, it appears that someone borrowed the remaining colluvial cover over the first approximately 250 feet of the tunnel in-by the original portal location. In this reach, the back (i.e., the roof) of the tunnel is now mostly gone and the tunnel is partially filled with damaged and displaced timber supports and what is assumed to be displaced colluvium ranging from silty sand to cobbles and boulders. The upper end of the now “collapsed, open” portion of the tunnel is blocked by a boulder at least seven (7) feet in visible dimension and water currently begins emerging at the surface at this point. It is inferred that the next approximately 70 feet of the tunnel upgradient in the colluvial section is at least partially plugged with displaced colluvium and broken timber supports. This reach is referred to as the “debris plug”. Recent geophysical profiling suggests that at least some of the remaining approximately 60 feet of the tunnel in the colluvial reach to the contact with Hermosa Formation bedrock remains open.

The key topographic and subsurface conditions described above are illustrated on the plan and profile views of all six alternatives as shown on Exhibits 4.1 through 4.6. The geologic conditions are best seen on Figures 5.1 and 5.2 in the Adit and Portal Investigation Report – 2013 Update (Atlantic Richfield Company, 2013).

3.0 Design Criteria

Key design criteria applicable to all alternatives include, but are not necessarily limited to:

- Accommodate peak flows up to the maximum discharge of 5.2 cfs (2300 gpm), based on historic data and 60 years of simulated flows at DR-3 from the Rico Site Underground Workings Hydraulic Model.
- Wherever feasible, utilize open channel conveyance of SLT discharge flows in preference to pipe flow to facilitate observation of conditions and clearing of obstructions when necessary.
- Where pipes are necessary to convey SLT discharge flows, design piping to resist scaling and eventual clogging, and provide appropriate redundancy of piping conveyance where necessary.
- Provide access to all piping for jetting and/or pigging equipment to control scaling.
- Provide access to open channels to remove any obstructions (ice/snow, beaver dams, tree limbs/branches, etc.), precipitation solids, or sediment that may accumulate over time.
- Minimize seepage losses from constructed conveyances (piping or open channels).
- Design life for all components is nominally 50 years.
- Provide for monitoring of water head in the St. Louis Tunnel, surface water flow, depth to groundwater, and surface and groundwater quality as appropriate to each alternative.
- Accommodate increasing hydraulic head if encountered over project life to provide for adequate flows by future designed additions or changes to the system if/as needed.

4.0 Description of Alternatives

A total of six (6) action alternatives were developed to address the objectives listed in Section 1.2 and meet the design criteria presented in Section 3.0. These alternatives are intended to cover an appropriately wide range of approaches to meeting the objectives and design criteria, with resulting differences in the associated constructability, operational functionality, risk, and long-term maintenance. Each of the alternatives is described in this section, including major components, their function, and key aspects of construction and operation.

4.1 Alternative 1 – Base case

Alternative 1 is referred to as the "Base Case" because it intentionally involves the least modifications to the existing conditions commensurate with addressing the objectives identified in Section 1.2. This includes leaving the existing debris plug in place and improving the collection and conveyance of water leaving the debris plug. In order to meet the objective of gathering to the extent practicable and conveying to treatment essentially all of the tunnel discharge, this alternative relies on the interpretation of available data that losses of tunnel discharge flows to the colluvium out-by the rock portion of the SLT and from the debris plug are minor and acceptable.

4.1.1 Components and function

The major components of Alternative 1 are illustrated on Exhibit 4.1, listed in Table 1.1, and described (in terms of both form and function) as follows:

Existing debris plug. The existing debris plug inferred present in the colluvial portion of the SLT will continue to convey tunnel discharges from the underground to the surface. The debris plug is assumed to be approximately 70 feet long, comprised of a heterogeneous mixture of broken and displaced timber supports (posts, beams and lagging) and colluvium. Based on hydraulic modeling described in Section 5.0, the bulk hydraulic conductivity of the debris plug is estimated as on the order of 3.9 cm/sec (equivalent to clean, open work gravel). Care will be taken during construction and subsequent operation to avoid to the greatest degree possible any disturbance of the debris plug and the overlying remaining colluvial cover.

AT-2 casing. The existing steel casing installed in boring AT-2 is approximately 22 feet long and extends from about 2.3 feet above the existing ground surface to what is inferred as an intersection with the north wall of an apparently open reach of the SLT. The internal diameter of the casing is 4 inches; a 2-inch PVC pipe is currently in place inside the steel casing to facilitate the measurement of water levels and extraction of water quality samples. The AT-2 casing will continue to function as a tunnel water level monitoring and water quality sampling point. It will also serve as a "relief well" to discharge tunnel flows if the permeability of the debris plug decreases either suddenly or gradually over time by internal collapse or clogging, and during times of sufficiently high flow from the mine workings as discussed in Section 5.0, resulting in further back-up of water in the SLT and interconnected underground workings. Grading would be improved so that water discharged through AT-2 would flow toward the wingwalls and be directed to discharge into the improved channel below the debris plug as described below.

Relief well(s). If plugging and resultant increased back-up of head above the debris plug occurs and is not adequately mitigated by drainage through AT-2, then one or more additional relief wells of appropriate capacity would be installed in the vicinity of AT-2 as described in more detail under Alternative 2.

Wingwalls. Concrete wingwalls will be constructed at the downgradient end of the debris plug and extend laterally to the toes of the slopes in the lower reach of the terrain trap. The walls will be connected to the upper end of the concrete channel described below and extend a minimum of 10 feet below existing grade, or to intersect an apparent groundwater perching layer if present and at shallow enough depth to be constructible (assumed as not more than 15 feet below grade given the nature of the colluvial subgrade). These wingwalls are intended to intercept water that either currently is, or in the future may, leak laterally into more pervious zones of colluvium from the debris plug or the upgradient remaining colluvial reach of the SLT. Given the inferred very heterogeneous nature of both the debris plug and the colluvium, it is not possible to accurately predict future changes in their hydraulic conductivity that might occur due to disturbance (e.g., earthquake shaking) or geochemical plugging (i.e., build-up of precipitates eventually clogging voids and pore spaces within these porous media), and where leakage might occur as a result. The wingwalls are located, aligned and extended to a depth to provide the opportunity to intercept such lateral leakage to the extent practical. A vertical zone of appropriately filter-protected drain rock with a longitudinal PVC drain pipe would be placed at the back face of the wingwalls to induce any seepage flow intercepted to rise to near the surface at the back of the wall. The PVC drain pipe and the surface grade at the back of the walls would be sloped to drain toward the new concrete channel and provision made for any such seepage to be discharged to the channel. If vertical or sufficiently steeply downslope-dipping higher hydraulic conductivity conduits are present in the colluvium, then some of the leakage could bypass the wingwalls.

Channel improvements. The existing timber support and colluvial debris in the collapsed, open portion of the SLT below the debris plug will be removed to the original tunnel floor grade. Special precautions will be necessary during excavation of the debris given the loose nature of the colluvium, depth of the channel, and proximity of especially the south slope to the excavation. Either an open concrete-lined channel or reinforced concrete pipe would then be constructed within the open channel. The selection and sizing of the conveyance in this reach will be optimized during final design considering safety and constructability, and long-term accessibility for monitoring and maintenance. At the preliminary design stage it is conservatively assumed that the resulting open channel will be lined with a concrete floor nominally 9 feet wide and vertical concrete walls extending approximately 8 feet to at least one foot above existing adjacent grade, and structurally designed to withstand the excavation wall and slope surcharge earth loadings. The new conveyance section, whether pipe or channel, will be fitted in a manner yet to be determined to the wingwalls and the downgradient end of the debris plug to ensure capture of the tunnel flows discharging from the plug and shallow leakage (if any) intercepted by the wingwalls. The new conveyance will extend to the location of the original portal (at the CMU block lime addition structure) for a total length of approximately 200 feet. The existing structure will either be modified or removed to facilitate conveying the tunnel discharges to a smaller, shallower trapezoidal concrete- or membrane-lined channel to the point of treatment (assumed here at the inlet to the demonstration wetland at Pond 19). Concrete lining is assumed at this preliminary design stage. This smaller channel is estimated at approximately 250 feet long.

4.1.2 Construction

Temporary measures will be implemented during construction to provide for the safety of workers and equipment from rocks rolling down the steep slopes in the terrain trap and potentially from the slope on the south side of the collapsed, open reach of the SLT. Such measures will likely include:

- Temporary barriers (e.g., concrete jersey barriers, braced steel panels, reinforced chain link fencing)
- Grading to reduce slope inclination above the work (potentially practical on a portion of the slope south of the collapsed, open reach of the SLT; possibly integrated with excavation at the SSR-A Phase I solids repository site)
- Minimizing the time workers and equipment are in the terrain trap to the extent feasible; exclusion of workers from the south side of the open collapsed reach of the tunnel unless fully protected from slope instability or rolling rocks
- Full-time observers watching for any evidence of slope instability (movement of cobbles or boulders, opening of cracks on or above the slope, erosional headcutting, etc.) during work periods when workers or equipment are exposed to potential harm (especially following any precipitation events or an earthquake)

Shut-downs during and following rainfall and snowmelt events and earthquakes until slope conditions can be assessed by competent persons and found safe to resume work

4.1.3 Monitoring

The function of Alternative 1 would be monitored for relevant conditions at an appropriate frequency to ensure that sudden or gradual decrease in the hydraulic conductivity of the debris plug, with the resultant increase in the head and storage volume of tunnel water, is observed in a timely manner to allow appropriate mitigation to be implemented. The monitoring would include, but is not necessarily limited to:

- Water head upgradient of the existing debris plug in AT-2
- Water flow in the conveyance below the existing debris plug (potentially covered by an upgraded sampling station DR-3)
- Water quality, on an if/as-needed basis, above the existing debris plug in AT-2
- Water quality in the conveyance below the existing debris plug (or potentially covered by an upgraded sampling station DR-3)
- Groundwater level and quality in selected monitoring wells, as necessary as part of the overall site monitoring plan

It is assumed that water head in the tunnel and flow in the conveyance channel below the debris plug would be collected essentially continuously utilizing permanently installed and routinely maintained automated data collection equipment fit for the purpose. It is also assumed that this data would be: 1) accessible from appropriate off-site locations; 2) programmed to send an alarm condition when head or flow is outside established ranges; 3) monitored by trained, competent staff at an appropriate frequency (assumed at least weekly for an initial period of operation up to one year, and then monthly); and 4) reviewed by a qualified professional at least quarterly. Increase in head above a trigger level to be established would initiate action to mitigate the head build up.

4.2 Alternative 2 – Base case plus relief wells

Alternative 2 includes all of the components and functions of Alternative 1, plus installation of two (2) new relief wells. Although the existing casing in AT-2 would be retained as a monitoring well, the analyses and design will not include AT-2 as a relief well in this alternative. During final design consideration can be given to considering AT-2 as at least a partially redundant well to the two (2) new relief wells. The following discussion addresses only those additional aspects of Alternative 2 that are different than Alternative 1.

4.2.1 Components and function

The major components of Alternative 2 are illustrated on Exhibit 4.2 and listed in Table 1.1. The components not already described under Alternative 1 are described below.

Relief wells. Two (2) new primary and two (2) redundant relief wells would be installed within the lower terrain trap adjacent and/or just upgradient of the existing AT-2 casing. The design capacity of these wells was developed as discussed in Section 5.0. These wells would be drilled and initially completed with at least 6-inch ID steel casing. The borings would be inclined at approximately -30° from horizontal so as to intersect the SLT north wall approximately midway between the floor and back, and at an angle to minimize the potential of the bit following down the tunnel lagging and posts and not penetrating the wall. The ends of the casings would penetrate the tunnel wall approximately a foot. The inlet for these relief wells would be set at approximately elevation 8855 (nominally four feet above the tunnel floor).

During final design consideration would be given to including a slotted end fitting with a cap designed to allow high capacity inflow while preventing a jetting tool or casing scraper used to periodically clean the pipe from leaving the well casing and becoming trapped in the tunnel. This would preclude the use of a conventional non-retrievable pig to clean the pipes. If a conventional pig were to be used it would be sacrificial and left in the tunnel after leaving the end of the pipe. Alternatively, a retrievable pig may be able to be found or fabricated.

Following the initial installation of the casing, a sufficiently wide trench excavation, supported as necessary by steel trench boxes or custom fabricated steel plates and struts, would be excavated around the new relief well casings. The casings would be cut off and a fabricated sweep would be welded to the casing. The downgradient end of the sweeps for the two primary relief wells would be set at approximately elevation 8861, the lowest feasible invert elevation to limit head build-up in the SLT during high inflow events greater than the capacity of the existing debris plug (see discussion in Section 5.0). A redundant well(s) would be set at a slightly higher invert (not more than a foot) to be determined based on constructability considerations. The relief wells would discharge through a manhole to a new culvert pipe as described below.

Discharge culvert. An appropriately sized (currently assumed as 24-inch) reinforced concrete pipe (RCP) culvert would be installed to convey SLT discharge from the relief wells as gravity flow to the new conveyance at the downgradient end of the debris plug. The culvert would be installed in a conventional cut and cover pipe trench, and penetrate the left (north) wingwall below grade. The culvert will connect to manholes as described below at its upgradient and downgradient ends. At the downgradient end, the culvert will penetrate the left (north) wall of the new concrete channel as shown schematically on Exhibit 4.2.

Manhole. A 60-inch diameter concrete manhole approximately 10 feet tall will be installed at the intersection of the relief well sweeps and the RCP culvert. The manhole will provide access to jet, scrape or pig the sweep and inclined portion of the relief wells and the culvert. A second manhole of comparable size near the downgradient end of the main reach of culvert will lower the invert of the culvert so that access is maintained into the terrain trap. The buried culvert section extending from this manhole will penetrate the left (north) wall of the new channel and discharge excess SLT flows below the debris plug.

4.2.2 Construction

The same temporary measures as described for Alternative 1 would be implemented during construction of Alternative 2 to provide for the safety of workers and equipment from rocks rolling down the steep slopes in the terrain trap and potentially from the slope on the south side of the collapsed, open reach of the SLT.

It is apparent that installation of the relief wells, manhole and culvert as planned for this alternative will require very thorough planning in terms of access to the site of the work within the terrain trap, protection of the workers and work during construction within the terrain trap, and the means and methods of construction. Work must occur within the very limited space available in the terrain trap (further limited by the necessary installation of rolling rock and debris slide protection), and the constraints of the required location and angles of the relief wells to appropriately penetrate the tunnel and then be able to be plumbed into a manhole as currently envisioned. These considerations will be further addressed during final design should this alternative be selected for implementation.

4.2.3 Monitoring

The function of Alternative 2 would be monitored for relevant conditions in a manner generally similar as for Alternative 1. The monitoring would include, but is not necessarily limited to:

- Water head upgradient of the existing debris plug in AT-2 and in at least one of the new primary relief wells
- Water flow in the RCP culvert prior to its discharge into the new conveyance or in the conveyance below the existing debris plug (potentially covered by an upgraded sampling station DR-3)
- Water quality, on an if/as-needed basis, above the existing debris plug in AT-2 or in at least one of the new primary relief wells

- Water quality in the RCP culvert prior to its discharge into the new conveyance channel or in the conveyance channel below the existing debris plug (or potentially covered by an upgraded sampling station DR-3)
- Groundwater level and quality in selected monitoring wells, as necessary as part of the overall site monitoring plan

The frequency of data collection and the review and assessment of the data would be as proposed previously for Alternative 1. As for Alternative 1, increase in water head within the SLT above an established trigger level would initiate appropriate changes to the system.

4.3 Alternative 3 – Relief wells plus plugging tunnel

Alternative 3 is similar to Alternative 2 in that new relief wells would be installed within the terrain trap. However, Alternative 3 also includes placing an impervious plug (assumed concrete in this evaluation) upgradient of the existing debris plug to block future flows from passing through the debris plug and causing them instead to be conveyed downgradient via the new relief wells. This additional feature is intended to eliminate the effects of collapse or clogging of the existing debris plug as a potential failure mechanism, but may result in internal erosion (piping) of colluvium into the existing debris plug without appropriate mitigation as noted in Section 4.3.1 below and described in Section 6.0.

As described below, this evaluation concludes that installation of a sufficiently secure impervious plug in the SLT upgradient of the debris plug is most technically feasible in a vertical installation. Given that a fill pad would be constructed for this installation, it is then also planned to install the new relief wells for this alternative vertically.

4.3.1 Components and function

The major components of Alternative 3 are illustrated on Exhibit 4.3 and listed in Table 1.1. The components not already described under Alternatives 1 and 2 are described below.

Fill pad. As shown on Exhibit 4.3, a fill pad would first be constructed to approximately elevation 8890 in the lower terrain trap. At this elevation the fill would not encroach onto adjacent USFS land. The pad would be approximately 5,200 square feet (sf) to provide working room for a drill rig to install the new tunnel plug and the new relief wells. The total amount of fill to be placed would be approximately 3,800 cubic yards (cy). The downgradient slope of the fill pad would be inclined at 2.5H:1V to allow equipment access (winched as necessary) to the working surface at the top of the fill. Guard rails or anchored jersey barriers would be installed on the access ramp as a safety measure during equipment mobilization/demobilization and materials delivery to the working platform. The fill material will be borrowed on site; if possible. Ideally excess material from construction of the SSR-A Phase I solids repository to be constructed immediately to the southwest would be used as the borrow source. If necessary due to timing of the construction of these two projects, an off-site borrow would be used.

The proposed fill pad would induce stresses onto the crown of the tunnel in the inferred still open colluvial reach of the tunnel between the debris plug and the Hermosa Formation, and on the debris plug reach. Although these stresses would be less than the tunnel experienced prior to the early 1950s when the terrain trap excavation was made, the condition of the assumed timber support in the still open reach and of the material in the debris plug after 60 years is unknown. In order to minimize the induced stresses of the fill, consideration would be given during final design to replacing the lower portion of the earth fill with concrete flow fill as a more rigid mat to bridge over, and thereby distribute a portion of the overlying soil load away from, the tunnel. This would require terraced forming up the slope of the terrain trap to retain flow fill in horizontal lifts.

Following placement of the new SLT plug and vertical relief wells as described below, the fill would be temporarily removed to approximately elevation 8875 to provide a lower working platform from which to excavate the trench described below as part of installation of the conveyance from the relief wells. Upon completion of the well installations, the fill would be replaced to elevation 8890 to provide a buttress against internal erosion daylighting at the ground surface and heave of the underlying colluvium during times when high heads are present in the SLT behind the new plug.

Upon completion of the channel improvements previously described under Alternative 1, the north portion of the downgradient slope of the fill would be reconfigured as necessary to provide for future equipment access to the working pad at the top of the fill.

Tunnel plug. A new tunnel plug would be installed at the upgradient end of the existing debris plug as noted above. It is proposed to place the new plug by drilling three vertical borings from the initial working platform into the tunnel at approximately 2-foot centers transverse to the tunnel centerline. Hydraulic concrete of an appropriate mix design, including additives to achieve quick set, would be placed by tremie pipe from the floor of the tunnel upward. All three holes would first be drilled and two of the holes, together with the vertical relief well borings described below, would be used as monitoring points to track the spread and setting of the plug during tremie placement in the first hole. Placement would then proceed by tremie through the remaining two holes at the plug location. It is intended that the plug be nominally 10 feet long (i.e., in the direction of flow).

Consideration would be given during final design to first placing two sacrificial plugs on either side of the location of the permanent plug using the methodology described above. Then the permanent plug would be placed as a pumpable grout mix under sufficient pressure to penetrate openings likely present between lagging in the assumed timbered sidewalls of the tunnel and any openings that might be present in the sacrificial plugs. The feasibility of this approach will depend on the required length of the three plugs to accommodate the spread and set of the materials used, while ensuring that the portion of the tunnel upgradient of the permanent plug is still accessible ideally to installing the relief wells vertically.

In either case, an acoustic televiewer and sonic imager would be deployed in one or more of the vertical drill holes to assess in-tunnel conditions to the extent feasible prior to initiating the plug placement, and any appropriate revisions to the installation plan made.

The effectiveness of the plug would be assessed by measuring the decrease (and ideally the termination) of flow in the channel below the debris plug, and the rise of water into the monitoring points immediately upgradient that would be converted to relief wells as discussed below.

A means to provide protection against the piping of colluvium into the adjacent debris plug due to the locally high seepage gradients anticipated at the concrete plug is still under consideration. One possible solution is to provide a ballasted and filter protected drain at the accessible downstream end. This would not, however, protect against internal piping and reconfiguration of the soils near the debris plug, but would protect against exit of piped soils from the system. At this time the most promising potential mitigation to address internal piping appears to be grouting the existing debris plug for its full length as thoroughly as possible to effectively minimize the available storage for the soil fraction that otherwise would be transported from the colluvium to the debris plug voids. It is important to understand, however, that it may be found impractical to fully mitigate this risk given the inferred nature of the debris (timber supports, boulders, cobbles) potentially blinding off large voids from full penetration of the grout. Furthermore, the potential for especially high local gradients and high velocity flows would likely be greatest at the interface of the new concrete plug and the colluvium where full contact may not be achieved due to the remote placement required. If grouting of the debris plug and the periphery of the concrete plug were to be implemented, grouting would be performed through vertical grout holes drilled from the temporarily lowered fill pad discussed above. Very close grout hole spacing and a variety of mix designs would likely be required to overcome the challenges of grouting under the inferred conditions.

Relief wells. The borings that would be completed as relief wells for Alternative 3 would be drilled vertically from the available work platform described above. These borings would initially be used as monitoring points to assist in tracking the placement of the new concrete tunnel plug as described above. Three of these monitoring points would be completed with PVC casing of sufficient internal diameter to later allow installation of the permanent steel relief well casings and subsequent grouting of the annulus between the casings. At least one of the holes would be completed with casing of sufficient diameter to accommodate temporary installation of a submersible pump to control the back-up of drainage water in the SLT during the placement of the new tunnel plug and conversion of the monitoring points to permanent gravity relief wells. The relief well casings would be the same size and material as for the Alternative 2 relief wells and completed with the same type of end fitting. Based on preliminary hydraulic analyses summarized in Section 5.0, it is anticipated that two (2) new relief wells would be installed, together with one (1) additional well for redundancy. The larger monitoring point with the submersible pump would be converted last after at least two of the relief wells were sufficiently complete to discharge tunnel flows into the new RCP culvert as discussed below.

Once the vertical well casing installations are complete, the fill pad would be temporarily lowered as described previously. Working from this lower platform, the new relief well casings would be accessed in a temporary braced excavation as described for Alternative 2. The vertical well casings would be tapped or fitted with a tee at invert elevation 8870. The horizontal legs of the tees would then be plumbed into an end cap on the RCP culvert to be installed in a manner and for the same purpose as described under Alternative 2. With this configuration, clean-out access for the vertical relief well casing by jetting, scraping or pigging is available from the replaced final fill working platform. Jetting of the horizontal leg of the tee could be accomplished with a 90-degree fitting on the jetting head. An approximately 5-foot tall manhole would be installed between the horizontal and inclined reaches of the RCP culvert as shown schematically on Exhibit 4.3 to provide access to jet any solids or debris that might accumulate.

Wingwalls. The wingwalls previously described under Alternatives 1 and 2 would be realigned, made higher above existing grade, and structurally designed as necessary where they would retain any of the permanent fill in the terrain trap. These walls would otherwise be designed to serve their primary function of intercepting lateral seepage that might still occur into the colluvium from the SLT above the newly placed concrete plug.

4.3.2 Construction

The same temporary measures as described for Alternatives 1 and 2 would be implemented during construction of Alternative 3 to provide for the safety of workers and equipment from rocks rolling down the steep slopes or local debris slides in the terrain trap. In addition to these measures, a ditch will be maintained around the periphery of the fill to be placed in the terrain trap as it is being placed to provide further protection from rocks rolling/tumbling off the slopes.

4.3.3 Monitoring

The function of Alternative 3 would be monitored for relevant conditions in a manner generally similar as for Alternatives 1 and 2. The monitoring would include, but is not necessarily limited to:

- Water head upgradient of the new tunnel plug in at least one of the new relief wells
- Water flow in the RCP culvert prior to its discharge into the new conveyance or in the conveyance channel below the point that discharges from the RCP culvert enter the channel (potentially covered by an upgraded sampling station DR-3)
- Water flow in the conveyance channel below the existing debris plug but above the point that discharges from the RCP culvert enter the channel (to monitor for any flow that is bypassing the new tunnel plug)
- Water quality, on an if/as-needed basis, above the new tunnel plug in at least one of the new relief wells
- Water quality in the RCP culvert prior to its discharge into the new conveyance channel or in the conveyance channel below the existing debris plug (or potentially covered an upgraded sampling station DR-3)
- Groundwater level and quality in selected monitoring wells, as necessary as part of the overall site monitoring plan

The frequency of data collection and the review and assessment of the data would be as proposed previously for Alternatives 1 and 2. As for those alternatives, increase in water head within the SLT above an established trigger level would initiate appropriate changes to the system.

4.4 Alternative 4 – Interception wall

The primary objective of Alternative 4 as originally conceived was to provide positive capture of all SLT discharge in the event that ongoing field investigations and analyses were to conclude that seepage losses to groundwater through the colluvium were not acceptable due to the potential for ultimately conveying dissolved metals in excessive concentrations to the Dolores River. Three fundamental design/construction criteria for this alternative remedy were to: 1) avoid reliance on the existing debris plug for conveyance by placing a reasonably water-tight wall through the colluvium out-by the rock portion of the SLT; 2) minimize the back-up of water behind the wall in the SLT to the extent feasible; and 3) avoid excavation and resulting destabilization of the colluvium (or the need for yet another retaining wall to retain a new colluvial cut slope).

As discussed in Section 5.0, preliminary analyses indicate that this alternative is not feasible for the currently estimated bulk hydraulic conductivity (i.e., permeability) of the colluvium that would be present inside the interception wall and through which the tunnel discharge would have to flow vertically. As shown in Table 5.3, even backing up head in the SLT to beyond the Blaine Tunnel level (approximately 500 feet above the SLT invert at the debris plug), and assuming what is judged the high end of the estimated range of bulk hydraulic conductivity for the colluvium, would barely convey the average SLT discharges measured historically at DR-3 or predicted in the mine workings hydraulic model. However, in the event that the hydraulic conductivity of the colluvium is ultimately judged to be sufficiently high (e.g., perhaps by the presence of several local highly conductive conduits within the colluvium), this alternative is described below. More detailed conceptual design would be completed only if this alternative appeared feasible based on the ongoing field investigations and associated analyses.

4.4.1 Components and function

The major components of Alternative 4 are illustrated on Exhibit 4.4 and listed in Table 1.1. The components not already described under Alternatives 1 through 3 are described below.

Interception wall. As shown on Exhibit 4.4, the primary objective and key design/construction criteria noted above for Alternative 4 are best met by constructing a three-sided box through the colluvium. This box would be tied into bedrock on all three sides, and extend vertically a sufficient height to provide for drainage of the SLT discharge vertically up through the colluvium, without having to excavate deeply into the colluvium to then drain the discharge downgradient of the wall. Although not fully optimized, the location and height shown on Exhibit 4.4 are a reasonable compromise to limit the overall size of the buried wall.

It would be necessary for this alternative to construct a wall that is adequately water-tight through colluvium that is known to locally contain large cobbles and boulders. Several placement methods were considered, including driving sheet pile, excavating and backfilling a trench with concrete by the slurry wall method, and drilling and placing the barrier as a concrete secant wall. The latter option is envisioned as the only practical method given the site topography and ground conditions.

Interception conveyance channel. Assuming that the colluvium inside the interception wall can convey the SLT discharges to the top of the wall, those flows would be directed to a lowered trapezoidal section cut into the top of the secant wall over the SLT alignment. This cut would be integral with a trapezoidal concrete–lined interception conveyance channel placed into shallow fill and/or colluvium on the existing slope between the top of the wall and the upgradient end of the channel improvements. This cut in the interception wall and the sloping channel will function as the inlet crest and chute of what would be, in effect, an overflow spillway to carry the SLT flows to the new channel and ultimately to the point of treatment. The inlet crest would be designed as a weir to facilitate flow monitoring as noted in Section 3.4.3 below.

Monitoring well. A monitoring well would be installed from the temporary working platform through the colluvium and bedrock into the downgradient reach of the rock portion of the tunnel as shown on Exhibit 4.4.

The well would be drilled and cased to accommodate a nominal 6-inch PVC outside casing with two 2-inch PVC pipes inside. The pipes and casing would be removed as the temporary fill is lowered and a surface completion made at the original ground. A retrievable transducer would be installed in one of the 2-inch PVC interior casings with cabling to a location outside of the terrain trap. Sampling would be performed in the second 2-inch PVC casing if/when required.

4.4.2 Construction

Installation of the secant wall would involve initially constructing a fill with a working pad at elevation 8930 (encroaching temporarily on USFS land as shown on Exhibit 4.4). A caisson rig would be used to drill vertical, steel-cased holes through the colluvium and nominally 2-3 feet into the underlying bedrock in a leap-frog manner. Every other hole would be spaced so that an intermediate hole would overlap the flanking holes when drilled. The first two holes would be filled with concrete to the top of the proposed secant wall, and by sand above that to the top of the working pad, as the steel casings are retracted. With the concrete still green, the intermediate hole would be drilled, cased and backfilled with concrete. A reasonably water-tight wall would result by repeating this process sequentially. A significant advantage of this method is that a coring bucket can be used to advance through boulders if/as necessary, and into bedrock at the bottom of the holes. Note that special measures not yet developed would be required to deal with the void where the secant wall crosses the inferred open colluvial portion of the SLT.

Other elements of the construction, including protection from rocks rolling/tumbling down the terrain trap slopes and local debris slides, would be comparable to those previously discussed for Alternatives 1 through 3 as applicable.

4.4.3 Monitoring

The function of Alternative 4 would be monitored for relevant conditions in a manner generally similar as for Alternatives 1 through 3. The monitoring would include, but is not necessarily limited to:

- Water head upgradient of the interception wall in a new monitoring well to be installed at the downgradient end of the rock portion of the SLT
- Water flow over the interception discharge channel inlet weir prior to its discharge down the chute and into the new conveyance channel
- Water flow in the conveyance below the existing debris plug (optional, temporary monitoring to identify if unanticipated significant leakage past the secant wall is occurring)
- Water quality, on an if/as-needed basis, above the interception wall in the new monitoring well
- Water quality in the conveyance channel below the existing debris plug (or potentially covered by an upgraded sampling station DR-3)
- Groundwater level and quality in selected monitoring wells, as necessary as part of the overall site monitoring plan

The frequency of data collection and the review and assessment of the data would be as proposed previously for Alternatives 1 through 3. As for those alternatives, increase in water head within the SLT above an established trigger level would initiate appropriate changes to the system.

4.5 Alternative 5 – Tunneling

Alternative 5 envisions tunneling through colluvium and bedrock beneath the terrain trap to intersect the SLT in what is anticipated to be good quality rock just beyond the reach noted as lagged in the available historic documentation. The tunneled reach through colluvium will be lined with large-diameter steel casing. The reach through rock will either be supported by conventional means (shotrcrete, mesh and rock bolts or steel sets and lagging) or the steel liner will be extended through part or all of the rock reach. A concrete plug will be installed in the SLT just out-by the new tunnel intersection. Only limited surface access into the terrain trap is anticipated as required for this alternative during construction, and no access is anticipated as necessary during operation. This approach provides direct access to divert all of the discharge flows in the SLT by gravity through a permanent, impervious conduit to a new concrete-lined channel to treatment. Work within the open, collapsed reach of the tunnel downgradient of the debris plug is also avoided with this alternative.

4.5.1 Components and function

The major components/techniques of Alternative 5 are illustrated on Exhibit 4.5 and listed in Table 1.1. The components/techniques not already described under Alternatives 1 through 4 are described below.

Tunneling. Based on review of available subsurface exploration and laboratory testing data, combined pipe ramming, hand-mining, and rock tunneling techniques appear most applicable. This would involve first hydraulically ramming a large diameter, heavy wall casing (assumed as 96-inch diameter for this evaluation) with a hardened cutting surface on the leading edge through the colluvium (supported with hand-mining as needed), and then advancing through the Hermosa Formation sedimentary bedrock to an intersection with the SLT using conventional rock- and hand-tunneling techniques.

The large diameter steel casing would be launched from a pit constructed immediately downgradient of the terrain trap between the Soil Lead Repository and the existing collapsed, open portion of the SLT as shown schematically on Exhibit 4.5. The launch pit would be excavated into colluvium and locally into the adjacent Soil Lead Repository with vertical walls to minimize temporary encroachment into the Soil Lead Repository and best utilize the footprint area available. The vertical cuts would be retained utilizing soil nailing, soldier pile and lagging, or some other appropriate means as determined during final design. Any soil lead material excavated would be placed in the open portion of the Soil Lead Repository or possibly in the Phase 1 portion of the SSR-A solids repository to be constructed just to the south of the tunneling operations. Any perched groundwater that might be encountered in the bottom of the launch pit would be controlled by grading to drain to one or more pumped sumps. Upon completion of the excavation and installation of wall support for the launch pit, a concrete thrust block and pipe ramming equipment pad and a concrete launch pad with rails to guide the 96-inch casing would be installed. The block and pads would be securely anchored into the colluvial subgrade to provide the necessary reaction to ram the casing while preventing the block and pads from sliding.

The alignment of the new tunnel would be adjacent and sub-parallel to the north wall of the existing SLT. The invert elevation at the launch pit would be slightly lower than the invert of the existing SLT to better ensure achieving positive gravity drainage from the existing and new tunnel through the 96-inch casing. The casing is advanced by a hydraulic hammer operating at several hundred blows per minute. The hammer reacts against a large concrete thrust block installed in the launch pit. The casing section in the launch pit rests on guide rails attached to a concrete launch pad constructed into the floor of the launch pit. The thrust block and launch pad are securely anchored into the colluvial subgrade of the pit as necessary to provide the necessary resistance to sliding during the pipe ramming operation.

The face in the colluvial reach of the tunneling would be mined in relatively short rounds (likely on the order of only a few feet) and the muck removed from the bore by a pipe auger where feasible. If necessary, given the ground conditions encountered, an excavator with a remote-controlled arm and backhoe claw-bucket could be used instead to remove loose ground. If/as necessary to control running ground conditions at the face through the anticipated relatively loose colluvium, the ground ahead of the face will be grouted from within the already placed casing prior to the next one to several advances. Where technically feasible and safe, grouting in the lower terrain trap may be placed from the surface in addition to or in place of grouting through the face of the tunnel. The grouting would form a halo of sufficiently stable colluvium around the bore to prevent uncontrolled running of the ground into the bore. Once grouted, the material at the face would be mined as described above and the casing advanced by ramming. If applicable to the ground conditions, the bore would be mined to a slightly greater diameter than the outer diameter (OD) of the casing and a biodegradable lubricant injected to facilitate the pipe ramming operation.

Based on the data available to date, the risk of encountering large boulders up to 3-foot diameter is judged high, and may occur on average on the order of once per 50 lineal feet of the drive through the colluvium. If a boulder larger than can be augered or otherwise excavated and transported back to the launch pit is encountered, then hand mining using Damite (non-explosive chemical fracturing), hydraulic fracturing, or a remote-operated hoe-ram mounted on a backhoe excavator would be employed to break down the oversize rock to a size that can be mucked by the auger. Boulders up to as large as 7-8 feet (as encountered within the terrain trap at the end of the debris plug and during drilling of BAH-01) are judged able to be handled with the proposed ramming method and 96-inch diameter casing by employing hand-mining techniques if/as needed. The alignment and grade of the casing would be maintained within tolerances (yet to be established) primarily by taking care not to over-push if a large boulder is encountered. In such cases the obstruction would be cleared by hand mining and/or alternative tunneling methods employed to keep the new casing on alignment and grade.

As the casing is advanced through the colluvium, new lengths would be welded to the pipe already in the ground. The welded casing will provide a water-tight conveyance to deliver SLT discharge flow to the new concrete-lined channel to be constructed downgradient of the existing debris plug as described for Alternatives 1 through 4. Pipe material and coating would be selected during design to accommodate the chemistry of the mine discharge water and meet the intended design life.

The rock portion of the new tunnel would be mined using chemical/hydraulic fracturing or a hoe-ramming machine, together with other hand mining techniques if/as necessary. Depending on the rock conditions anticipated based on existing and ongoing field exploration, either the steel casing will be advanced through some or all of the rock portion of the alignment or the rock will be supported with fiber-reinforced shotcrete, mesh and rock bolts or steel sets and lagging.

When the new tunnel is close to breakthrough into the existing SLT the water in storage behind the debris plug will be drained by drilling a pattern of sequentially lower small diameter holes through the remaining rock starting near the elevation of the back of the SLT. If feasible, a submersible pump will be placed into the tunnel through the first drain hole and stored water in the tunnel pumped down to reduce the subsequent water handling requirements. The remaining flowing mine discharge water exiting each lower drill hole in turn will be routed into a temporary pipe or allowed to flow on the floor of the tunnel/invert of the casing to a sump and pump arrangement in the launch pit. It is envisioned that these flows would be conveyed by temporary piping during construction, or the new conveyance channel constructed early to serve the water handling needs during construction. Using this methodology will control the head (and thereby the flow rate) of stored mine water to be handled at one time to more manageable levels. It is assumed that the debris plug will continue to convey most of the SLT discharge flow during this unwatering operation. Upon completion of the unwatering of the stored water in the SLT, the remaining rock will be mined to breakthrough into the SLT. It is currently envisioned that this will be a hand-mining operation as the alignment of this last short portion of the new tunnel will diverge significantly from the primary alignment to intersect the SLT at a more favorable angle.

Tunnel plug. Once the breakthrough into the SLT is accomplished the full discharge flow will be routed through the new tunnel/casing to the launch pit and from there conveyed to the new conveyance channel adjacent to the debris plug. The diversion will be accomplished by placement of a temporary cofferdam on the floor of the SLT at the downgradient side of the breakthrough. With the SLT discharge flow diverted, a concrete tunnel plug will be placed just downgradient of the temporary cofferdam. It is currently envisioned that the plug will be constructed of reinforced concrete doweled into the floor, walls and back of the SLT. Any joints, fractures or other openings in the rock around the tunnel periphery will be sealed prior to placement of the plug. The plug length is assumed as a minimum of 5 feet to ensure a water-tight seal. With the new tunnel plug placed downgradient of the intersection of the new tunnel with the existing SLT, access into the remainder of the SLT will be available assuming conditions are safe or made safe.

Casing appurtances. An elevated steel grate walkway would be installed approximately 1.5 feet above the invert of the 96-inch steel casing to provide access to the full length of the new drainage tunnel, and potentially into the St. Louis Tunnel depending on conditions observed during construction. Steel grating with a lockable access door would be installed at both ends of the casing to prevent unauthorized entry into the casing and the SLT while allowing unimpeded discharge of SLT flows.

Conveyance channel. Upon completion of the tunneling portion of the work most of the launch pit will be backfilled with well-compacted granular borrow material. As shown schematically on Exhibit 4.5, a trapezoidal concrete-lined conveyance channel will be constructed from the downgradient end of the 96-inch casing to the vicinity of the original SLT portal and beyond to treatment as for Alternatives 1 through 4. Consideration will be given during final design to constructing the conveyance in the deeper reach adjacent to the existing open, collapsed portion of the SLT in a RCP installed by conventional cut and cover pipe trench construction.

Collapsed SLT backfill. The existing collapsed, open portion of the SLT from the downgradient end of the debris plug to the original portal location will be cleared of remnant and broken timber supports and large boulders to the extent that their removal can be done safely without destabilizing the adjacent slope to the south. This reach will then be backfilled with nominally compacted granular material to remove the existing safety hazard and stabilize the toe of the adjacent colluvial slope to the south. All work will be done remotely with conventional or long-stick equipment for debris removal, fill placement, and compaction (utilizing a plate compactor on a trackhoe boom).

4.5.2 Construction

The construction of the tunneled portion of Alternative 5 is described above as part of the discussion of the components and functioning in Section 4.5.1. This alternative will require that appropriately trained contractor personnel experienced with tunneling/mining operations work underground. It will also be necessary that a highly experienced tunneling expert familiar with the design and anticipated ground conditions be available during the pipe ramming and conventional and hand-mining operations to assist in assessing and appropriately reacting to actual ground conditions encountered. Given the known and anticipated ground conditions, the recommended strategy is to plan for and have on-site an appropriate suite of equipment, materials and tools to immediately respond to challenges that might arise.

A key element of the safety measures recommended during construction will be frequent observation and detailed remote LIDAR survey of the ground, and/or mounted surface survey targets in the vicinity of the bore during pipe ramming through the colluvial material. Consideration will be given to installation of movement detectors installed in shallow inclined borings in the lower slopes of the terrain trap to provide warning of larger slope movements than the surficial raveling that occurs more or less continuously (e.g., ShapeAccelArray (by Measurand, Inc.). Vibration monitoring will also be installed at the surface around the toe of the terrain trap slopes to assess loadings during pipe ramming.

Other elements of the construction, including protection from rocks rolling/tumbling down the terrain trap slopes, would be comparable to those previously discussed for Alternatives 1 through 4 as applicable. However, the exposure of workers to such conditions are anticipated to be substantially less as the only access to the terrain trap envisioned during construction will be during the installation and servicing (if/as needed) of safety instrumentation.

4.5.3 Monitoring

The function of Alternative 5 would be monitored for relevant conditions in a manner generally similar as for Alternatives 1 through 4. The monitoring would include, but is not necessarily limited to:

- Water flow in an appropriately designed weir in the conveyance channel below the downgradient end of the 96-inch steel casing (or potentially covered by an upgraded sampling station DR-3)
- Water quality, on an if/as-needed basis, in the conveyance below the downgradient end of the 96-inch steel casing (or potentially covered by an upgraded sampling station DR-3)
- Groundwater level and quality in selected monitoring wells, as necessary as part of the overall site monitoring plan

The frequency of data collection and the review and assessment of the data would be as proposed previously for Alternatives 1 through 4.

4.6 Alternative 6 – Retaining wall

The primary objective of Alternative 6 as originally conceived was to intercept the SLT via a supported open cut in what is anticipated to be good quality rock just beyond the reach noted as lagged in the available historic documentation. This approach provides direct access to divert all of the discharge flows in the SLT by gravity (in a concrete channel or pipe) through the terrain trap and open, collapsed portion of the SLT to a new concrete-lined channel to treatment generally as described previously in Alternatives 1 through 4. Three fundamental design/construction criteria for this alternative remedy are to: 1) avoid reliance on the existing debris plug for conveyance by placing a water-tight plug anchored into competent rock within the SLT; 2) minimize the back-up of water behind the plug by use of a new concrete channel (or concrete pipe in some reaches) to convey SLT discharges to treatment; and 3) provide the potential to gain access to the SLT in the future if conditions in the tunnel allow.

4.6.1 Components and function

The major components of Alternative 6 are illustrated on Exhibit 4.6 and listed in Table 1.1. The components not already described under Alternatives 1 through 4 are described below.

Access ramp/working platform. Fill would be placed within the terrain trap and extending into the corridor between the collapsed, open portion of the SLT and the Soil Lead Repository to provide construction access to a working platform from which the micropile retaining wall described following would be installed. The base of this fill will be flow fill placed behind an earthen starter dike; the remainder of the fill to elevation 8940 will be compacted earthfill. A mechanically stabilized earth retaining wall would be constructed to contain the fill on the north side to minimize encroachment on the Soil Lead Repository. Guard rails or anchored jersey barriers would be installed on the access ramp as a safety measure during equipment mobilization/demobilization and materials delivery to the working platform.

Micropile retaining wall. As shown on Exhibit 4.6, the primary objectives and key design/construction criteria (noted above in Sections 1.0 and 3.0, respectively) for Alternative 6 are met by constructing a three-sided retaining wall that would form a corridor providing gravity drainage of SLT discharges to treatment and direct access for personnel and equipment to a new portal in rock at the SLT. The retaining wall would be installed through the colluvium and tied into bedrock. Several retaining wall options were considered, including sheet pile, soldier pile and lagging, a secant wall, and a reticulated micropile wall. A combination of a micropile wall with horizontal walers and struts is envisioned as the most practical method given the site topography and ground conditions.

Temporary bypass pipe. A temporary steel pipe may be installed in the open cut corridor to convey SLT discharges by gravity flow to the existing earthen channel downgradient of the work area. Alternatively, the new concrete conveyance channel or RCP described below may be constructed prior to breakthrough to the SLT to facilitate temporary management of the SLT discharge flows and subsequent installation of the new tunnel plug and portal appurtenances described below.

Tunnel plug. Once the excavation and braced micropile wall and the temporary bypass pipe or new channel are in place, then breakthrough into the SLT through the retaining wall will be accomplished in a manner similar to that described for breakthrough of the new tunnel into the SLT under Alternative 5. In this case, the breakthrough would be through the micropile retaining wall rather than native rock. With the SLT discharge flow diverted into the temporary bypass pipe or new channel or pipe, a permanent plug will be placed just out-by the breakthrough to prevent future flow through the downgradient reach of the SLT and the existing debris plug. Any joints, fractures or other openings in the rock around the tunnel periphery will be sealed prior to placement of the plug. The new tunnel plug will be constructed of reinforced concrete doweled into the floor, walls and back of the SLT. The bulkhead length is assumed as a minimum of 5 feet to ensure a water-tight seal.

Portal appurtenances. The penetration through the retaining wall into the SLT will be fitted with vertical steel grating doweled into the tunnel periphery with a lockable access door. The grating will pass the SLT discharge flows while preventing unauthorized access into the SLT. Authorized access to the portion of the tunnel in competent bedrock in-by the new tunnel plug (and further into the SLT if conditions permit) would be possible through the access door.

Conveyance channel. Flows through the portal grating would be directed to a trapezoidal concrete–lined conveyance channel constructed in colluvium in the bottom of the open cut corridor. The channel will extend to the point of treatment as for the preceding alternatives. A transition structure would be installed to direct the SLT discharges into the new trapezoidal channel. The temporary bypass pipe described above, if used, would continue to function until the new channel is ready to accept flow. If desirable, removable steel grating could be installed over the new conveyance channel to facilitate personnel and equipment access to the new tunnel portal. As noted previously, use of RCP would be further evaluated as an alternative to open channel conveyance during final design.

4.6.2 Construction

Installation of the microplie wall would involve initially constructing a fill with a working pad at elevation 8940. The fill will begin with the construction of a 20-foot high dike located at approximately Station 2+00 that would seal off the entrance of the terrain trap and act as a dam to retain flow fill that would be poured behind (i.e., upgradient of) the dike into the terrain trap to form a flat working platform. This first step to place the fill will help maintain water flow through the debris plug while the retaining wall is built and until SLT discharge flow is subsequently diverted. A 2.5H:1V earthen ramp and working platform will be constructed over the flow fill to elevation 8940 as shown schematically on Exhibit 4.6. The fill material will be borrowed on site; if possible. Ideally excess material from construction of the SSR-A Phase I solids repository to be constructed immediately to the southwest would be used as the borrow source. If necessary due to timing of the construction of these two projects, an off-site borrow would be used.

A micropile rig would be used to drill vertical, steel-cased holes through the fill and colluvium and nominally 2-3 feet into the underlying bedrock on a 5-foot spacing to form all three sides of the retaining structure (end wall and flanking tapered walls). Excavation of the material in between the end wall and tapered walls (i.e., the open cut corridor) will be implemented by a top down method. Material will be excavated in horizontal lifts on the order of 5 to 10 feet at a time. Shotcrete will be installed after excavation to prevent the soil between the micropiles from raveling and running. A waler (i.e., a horizontal beam on each flanking tapered wall) and horizontal struts spanning the opening between the tapered walls will then be installed. The excavation and installation of permanent retaining wall supports will continue in the same manner until reaching the bottom of the walls at approximately the SLT invert elevation. There are several key advantages to this method: 1) the micropile drill rig can be used to advance through boulders if/as necessary and into bedrock at the bottom of the holes; 2) by using the whalers and struts there is no need for long rock anchors penetrating the colluvium and into bedrock that are difficult to install and of less certain load-bearing capacity; and 3) the struts can be designed to support a roof over the open cut corridor if desired for additional long-term rockfall protection.

This alternative will require that appropriately trained contractor personnel experienced with micropile operations and heavy ground support in deep, open excavations are on-site during these operations. It will also be necessary that a highly experienced micropile and retaining wall expert familiar with the design and anticipated ground conditions be available during these operations to assist in assessing and appropriately reacting to actual ground conditions encountered.

Other elements of the construction, including protection from rocks rolling/tumbling down the terrain trap slopes, would be comparable to those previously discussed for Alternatives 1 through 4 as applicable.

4.6.3 Monitoring

The function of Alternative 6 would be monitored for relevant conditions in a manner generally similar as for Alternatives 1 through 5. The monitoring would include, but is not necessarily limited to:

- Water flow over the interception discharge channel inlet weir prior to its discharge into the new conveyance channel (optional)

- Water flow in an appropriately designed weir in the conveyance channel below the existing debris plug (or potentially covered by existing or upgraded sampling station DR-3)
- Water quality in the conveyance channel below the existing debris plug (or potentially covered by existing or upgraded sampling station DR-3)
- Groundwater level and quality in selected monitoring wells

The frequency of data collection and the review and assessment of the data would be as proposed previously for Alternatives 1 through 5.

5.0 Hydraulics

The following discussion in Sections 5.1 through 5.3 summarizes the methodology used to simulate hydraulic conditions to support the evaluation and conceptual design of alternatives presented in Section 4.0 that include flow in the existing debris plug (i.e., Alternatives 1, 2 and 3). The pre-debris plug condition is also modeled to provide context and perspective to the development and evaluation of the remedial alternatives under consideration. A discussion of key results from the hydraulic modeling is presented in Section 5.4. Potential use of the SLT to attenuate seasonally higher tunnel discharges by intentionally utilizing the available storage volume in the interconnected mine workings was also evaluated as discussed in Section 5.5.

An evaluation of the potential for and consequences of failure of the debris plug and/or colluvium out-by the rock portion of the SLT due to increase in head above the debris plug (and resulting geotechnical instability as discussed in Section 6.0) is presented in Section 7.0.

5.1 Underground hydraulics

The SLT acts as a drain for groundwater at and in the vicinity of the Rico-Argentine Mine Site. The network of interconnected underground workings in Telescope Mountain (and its lower slopes known as CHC Hill) and Dolores Mountain drain to the St. Louis (or 500) level and discharge from the SLT to the St. Louis Ponds System. Flow is measured at DR-3, downstream of the existing debris plug and collapsed, open lower section of the SLT as described in Section 2.0. Flow measurements at this location have occurred intermittently since the late 1970s and represent a small, incomplete dataset. Inflow values are critical to evaluation of the hydraulic conditions within the SLT and the overlying colluvium out-by the rock portion of the tunnel under different design alternatives.

To supplement the DR-3 dataset, a preliminary predictive hydraulic model (Rico Site Underground Workings Hydraulic Model) was utilized to estimate daily flows at DR-3. The model is driven by precipitation and snowmelt, and was calibrated using DR-3 flow measurements and Dolores River flow measurements. The model uses a water balance approach based on total available water, surface runoff, mine water runoff, and water lost due to physical processes (e.g., evapotranspiration). The predicted flows from this model are the best available data with which to analyze the hydraulics associated with different flow conditions at the tunnel debris plug under the different design alternatives presented in Section 4.0.

Figure 5.1 shows the predicted daily flows at DR-3 between 1951 and 2011. These daily inflow data were assumed to be representative of flows inside the SLT (i.e., negligible losses to colluvium are assumed as discussed below in Section 7.3) and were used as the inflow input to the flow routing model discussed next. The apparent truncation of modeled flows at approximately 5 cfs is the result of the physical limitations inherent in the model (and in the field) of the degree of infiltration and soil/fault/fracture “abstraction” storage of snowmelt and rainfall that can ultimately report to DR-3. In other words, there is a physical limit to the amount of precipitation (snowmelt or rainfall) that can enter the underground workings regardless of the amount of the amount of precipitation that occurs. The excess flows that would otherwise report to DR-3 run off to the stream systems instead and show up as higher stream flows. Note that the streamflows are not limited in the same way as the mine workings inflows so that years with sufficiently high precipitation (beyond the capacity of the mine workings to absorb) can and do show streamflow peaks that appear clipped in the mine workings model outflow.

5.2 Flow routing

A spreadsheet model was developed to route flow from the SLT (upstream of the debris plug) through different outlet conditions and to predict increase in head and accumulation of mine water within the underground workings, using the storage-indication level pool routing method. This method uses the stage-storage relationship and stage-discharge relationship to route flow. A stage-storage curve was developed for the 500 level from the tunnel portal through the SLT and the southeast (SE) and northwest (NW) cross-cuts using known elevations and assumed tunnel dimensions of 7 feet in height by 9 feet in width. A total of 46 feet of head is available within the modeled portion of the 500 level, which is equivalent to approximately 600,000 cubic feet (cf, or 13.8 acre feet) of storage (Figure 5.2). Head loss within the tunnel was assumed to be negligible. Evaluation of even greater head increase in the tunnel above the debris plug, and the accompanying accumulation of mine water, to the Blaine or 100 level was not pursued for the reasons discussed in Sections 5.4 and 5.5.

5.2.1 Rating curves

Discharge rating curves were developed for the different outlet conditions. The methods used to develop these relationships are described as follows.

Debris Plug:

The stage-discharge relationship for the current condition debris plug flow was estimated using Darcy's Equation:

$$Q = K\frac{\Delta H}{\Delta L}A$$

Where: Q = discharge (cfs)

K = hydraulic conductivity (ft/s)

ΔH = Change in height across debris plug (ft)

ΔL = Length of debris plug (ft)

A = Cross sectional area of debris plug (ft^2)

Hydraulic conductivity for the debris plug was back calculated using known water levels in AT-2 and known flow rates at DR-3. This calculation was performed for three manual water level measurements taken in March, April and June of 2013 at AT-2 and yielded an average value of 0.13 ft/s (3.9 cm/s).

Relief Wells:

Stage-discharge curves were developed for the relief wells by solving the energy equation and the Hazen-Williams equation simultaneously. The conservation of energy was solved for discharge as:

$$Q = A\sqrt{(H_1 - H_2 - H_L - H_{ml}) * 2g}$$

Where: Q = discharge (cfs)

A = flow area inside pipe (ft^2)

H_1 = upstream head, measured from tunnel invert (ft)

H_2 = distance to discharge point or top of casing, measured from tunnel invert (ft)

H_L = head loss (ft)

H_{ml} = minor head losses (ft)

G = gravity (ft/s^2)

The Hazen-Williams equation was used to calculate head loss as a function of discharge:

$$H_L = \left[\frac{\frac{Q}{A}}{1.318 * C * \left(\frac{d}{4}\right)^{0.63}}\right]^{1.85} * L$$

Where: HL = head loss (ft)

Q = discharge (cfs)

A = flow area inside pipe (ft^2)

C = roughness coefficient, 100 for steel

d = pipe diameter (ft)

L = pipe length (ft)

Table 5.1 shows the design parameters used for each of the alternatives.

5.2.2 Results

This section provides a brief overview of the hydraulic model output results shown in Tables 5.2, 5.3, and 5.4 and Figure 5.3. All head values are measured from the invert of the tunnel at the upgradient end of the inferred existing debris plug, with maximum heads and average heads quantified using the full 60-year period of record of simulated flows. Post 1999 maximum heads were quantified using analysis results from the time period when the current debris plug is inferred to have existed.

- **Alternative 0** represents conditions prior to the installation of AT-2 (or complete plugging or blockage of AT-2), with the only discharge occurring through the debris plug. The maximum head for this alternative was 44 feet, which represents the worst case condition modeled by allowing head to build up as high as necessary in the SLT and SE/NW cross-cuts. The post-1999 maximum head value of 41 feet represents the estimated worst case condition that the current debris plug has been exposed to.
- **Alternative 1** is the existing condition. Water discharges through the debris plug and AT-2 is utilized as a relief well when heads increase to its discharge invert. The maximum head under this condition was 29 feet, 15 feet less than for Alternative 0.
- **Alternatives 2a and 2b** assume that AT-2 is capped. The debris plug continues to transmit flow, and two new angled, 6-inch (2a) and 8-inch (2b) diameter relief wells are installed approximately 5 feet below the current grade (the hydraulics analysis does not consider any wells installed for system redundancy). Of the alternatives modeled, these alternatives had the lowest maximum head values of approximately 13 feet and 11 feet above the tunnel invert for 2a and 2b, respectively.
- **Alternative 3** assumes that the debris plug is sealed at its upgradient end and that 1, 2, or 3 new vertical, 6-inch diameter relief wells are installed and discharge at an elevation of 8870 feet. The minimum discharge head under this condition is approximately 19 feet above tunnel invert, as no

outflow occurs below this point, which raises the average head significantly under this condition compared to the other design alternatives. With 3 relief wells the maximum head experienced is 22 feet above the tunnel invert, only 3 feet above the minimum discharge point. This indicates that the relief capacity of this alternative is limited by the height of the discharge point.

- **Alternative 4** could not be modeled under the range of hydraulic conductivities estimated for the colluvium. The tabular rating curves for the colluvium using the high and low hydraulic conductivities are shown in Table 5.3. At 500 feet of head (which would result in discharge out the Blaine portal), the discharge would be less than one-third of the maximum inflow, rendering this alternative hydraulically infeasible for the conditions assumed and modeled. Only if the hydraulic conductivity of the colluvium was orders of magnitude higher, or one or more relief wells were included, would this alternative become hydraulically feasible.

In addition to the hydraulic analyses performed primarily to support conceptual design of Alternatives 1 through 3, estimates were made of the additional head and storage that would develop upgradient of the existing debris plug under Alternatives 1 and 2 if the debris plug hydraulic conductivity was reduced due to sudden collapse (perhaps due to earthquake shaking) or progressive blinding due to precipitation of sludges or coatings. These analyses assume that the casing at AT-2 for Alternative 1 and the two relief wells for Alternative 2 are present and fully functional. The results of these analyses are presented in Table 5.4. Note that the prior results for Alternative 0 assuming no blinding and no other flow conveyance than the debris plug are included in Table 5.4 for perspective. The results for Alternative 1 show that even a very minor reduction in the hydraulic conductivity of the debris plug causes a modest increase in the head for the average inflow case and a very significant increase for the maximum inflow case. This is due to the fact that the existing casing in AT-2 is undersized and cannot provide adequate hydraulic relief to mitigate blinding of the debris plug. On the other hand, the results for Alternative 2 show only minor additional head even when the debris plug is essentially fully blinded (i.e., when the hydraulic conductivity is reduced to 10^{-5} cm/sec). This indicates that the two relief wells are very effective assuming that they remain fully functional.

5.3 Flood frequency and probability analysis

Risk associated with the hydraulic results presented herein is dependent on the likelihood of the simulated peak events occurring, referred to as the peak discharge recurrence interval. For surface water drainage basins recurrence intervals are typically determined based on a Log-Pearson III probability distribution as recommended in Bulletin 17B by the Interagency Advisory Committee on Water Data (1982). As discussed previously in Section 5.1, discharge from the Rico SLT adit is controlled by the same hydrologic inputs (rainfall and snowmelt) as the surface water systems (i.e., Dolores River and Silver Creek), but is physically limited by the rate at which water is able to enter the underground mine workings through infiltration into overlying soil and faults/fractures. This limitation implies that peak discharge from the Rico underground mine workings cannot exceed some maximum threshold, which is not considered in surface water peak discharge probability analyses (i.e., Log –Pearson III). Therefore, it is clear that application of the Log-Pearson III probability distribution provides a conservative estimate of peak mine water discharge events. Using the simulated flow data from the Rico Site Underground Workings Hydraulic Model, peak discharges were estimated using the Log-Pearson III analysis for the 2-, 10-, 25-, 50-, and 100-year events as shown in Table 5.5. Based on this analysis, the peak flow estimated by the Rico Site Underground Workings Hydraulic Model corresponds to approximately the 10-year event.

An additional routing analysis was conducted using the modeled 25-, 50-, and 100-year runoff event hydrographs to conservatively assess the potential effects of infrequent mine water inflows. These hydrographs were constructed by scaling a peak runoff year selected from the 60 years of simulated inflows to match the peak flows for each desired recurrence interval. The constructed hydrographs and the peak year simulated hydrograph (approximately equivalent to the 10-year hydrograph) are plotted on Figure 5.4. The results (shown in Table 5.6) indicate that Alternative 0 requires a greater head then what is available in the 500-level (46 feet) to discharge the peak flows for all three events. The same is true under Alternative 1 for the 100- and 50-year events, while during the 25-year event there is a maximum head of 39 feet. Alternative 2b achieves the lowest maximum heads for all three events. There is only a difference of one foot in maximum heads during the 25-year and 100-year events under this alternative, indicating that two 8-inch relief wells are able to accommodate even the most conservative maximum flow rates, and are only limited by the invert of the discharge point (i.e., top of casing).

5.4 Discussion

The model results presented indicate a wide range of maximum head values under the different design alternatives for the 60 years of simulated adit flows. The largest maximum head occurred for Alternative 0, where the debris plug is the only means of outflow from the SLT. Figure 5.5 shows the yearly maximum predicted inflow and the resultant maximum head value for the modeled 60-year period. This indicates that flow rates approximately equal to the period of record maximum (approximately 5 cfs), occurred during 15 of the 60 model period years or 25 percent of the time. It is important to note, however, that these are simulated flow rates that have been calibrated to a relatively small set of actual flow measurements as discussed above.

Figure 5.6 shows measured flow rates at DR-3 between 2000 and 2007 (after collapse of a portion of the SLT in colluvium and prior to installation of the casing at AT-2) and the equivalent head upstream of the debris plug based on Alternative 0 conditions. These data indicate a maximum head of 22 feet upstream of the debris plug. Although these measurements were intermittent and the Rico Site Underground Workings Hydraulic Model predicts larger flows to have occurred during this period (indicated by the 41-foot post-1999 maximum head value for Alternative 0), it is conservative to assume that the debris plug has not experienced heads equivalent to the estimated maximum for the 60 year period of record modeled.

For alternatives that include continuing discharge through the existing debris plug, the head behind the plug under modeled average inflow conditions is anticipated to rise to elevation 8860 for Alternative 2 and to elevation 8873 for Alternative 3. Heads behind the debris plug for these two alternatives under modeled maximum inflows are anticipated to rise to elevations 8864 and 8880, respectively. Based on these results, Figure 5.7 shows the estimated maximum limits on the slopes out-by the SLT where losses laterally through bedrock discontinuities and the overlying and adjacent colluvium could result in intermittent seepage, development of new springs, and possibly local slope instability that could then progress upslope. Such effects would be anticipated to be more likely during times of high inflows and at locations where interconnected higher hydraulic conductivity pathways emerge on the slope face (pathways that are very likely present but not feasible to locate prior to seepage emerging, given the geologic conditions at the site). If significant blinding or plugging of the debris plug occurs then even greater increases in head and accumulation of mine water would occur. This could lead to local areas on CHC Hill and in the Silver Creek drainage potentially subject to these effects extending as high as approximately 500 feet above the SLT invert at the debris plug (approximately elevation 9340), with drainage from the Blaine portal (see Figure 5.7 for a conceptual idea of the potentially impacted area on CHC Hill).

The probability analysis and associated flow routing estimate the hydraulic performance of each of the design alternatives depending on the continued functioning of the debris plug under extreme events, as predicted through conservative analyses. These results provide important insight for the alternatives that rely on relief wells to discharge flows, as the system's performance is largely dependent on the invert discharge elevation of the relief wells. Further discussion of the risk associated with these infrequent events is provided in Section 7.0.

Based on the results of the hydraulic analyses discussed above, an evaluation of the geotechnical risks and possible failure mechanisms associated with the debris plug and adjacent and overlying colluvium under a range of conditions is presented in Section 6.0 following.

5.5 High flow attenuation

An assessment was made of the potential benefits versus negative consequences of utilizing the estimated available storage space in the underground mine workings to attenuate seasonally high mine inflows. If safe and technically feasible, such attenuation could theoretically reduce the maximum design inflow for treatment of SLT discharges or reduce the frequency and size of flows that would bypass treatment. Any such benefits would have to be weighed against the potential implications of intentionally greater temporary accumulation of water in the mine workings with the associated increased heads (see Figure 5.7). These implications could include: 1) uncontrolled seepage discharge to the face of CHC Hill; 2) destabilization of colluvium and existing landslide deposits on CHC Hill; 3) increased hydraulic stress on the debris plug and 4) wetting and possibly increased dissolution of metals from the tunnel walls over what would occur otherwise. To assess the potential benefit of mine storage attenuation, an evaluation was conducted to determine the feasibility of storing seasonal peak flows, during an average flow year, in the underground mine workings to match a maximum outflow rate.

This assessment used a rudimentary water balance to calculate storage volume based on the difference between inflow and outflow. The outflow was arbitraily limited to 1000 gpm, or 2.2 cfs to illustrate the concept under evaluation. Figure 5.8 shows the average daily inflow hydrograph (calculated based on the 60 years of simulated inflow data), the outflow hydrograph, and the volume of water in excess of 1000 gpm (represented by the cross-hatched red area). As this flow rate was exceeded by the inflow hydrograph, the accumulated storage was calculated using the excess flow rate, and as the inflow dropped below the maximum outflow, the available storage was discharged, limited to a total of 1000 gpm when combined with the inflow hydrograph. This would be achieved by an adjustable valve to maintain a constant maximum flow rate, independent of the upstream head. Equivalent head values within the underground mine workings were determined using an order-of-magnitude-level stage-storage curve, extended from the curve used in the routing analysis discussed in Section 5.2, to include the potential use of storage available in the Blaine-Argentine Mine workings above the 500 level to, but not including, the 100 level (approximately 500 feet above the invert at the debris plug). Discharge from the Blaine portal (at the 100 level) would be directly to Silver Creek without additional hydraulic controls at that location, and therefore was excluded from the evaluation. The workings to the north of the SLT (referred to herein as the Mountain Springs-Wellington workings) are at a higher elevation than the Blaine portal and thus would not provide for further accumulation of mine water before discharge from the Blaine portal would occur.

Figure 5.9 shows the accumulation of storage (and equivalent head) in the mine workings based on the water balance. Storage begins accumulating in mid-June and within one month the mine workings up to the Blaine level are full (as indicated by the dashed red line on Figure 5.9). Storage continues to accumulate through mid-August, at which point the excess storage begins to discharge and is completely discharged by mid-October. The 1000 gpm outflow rate used in this evaluation was not based on any known or desired condition, but was rather used as a general condition to demonstrate the practicality of using the full storage up to the Blaine Level to control outflows. Under the conditions presented, the inflow exceeded the outflow restriction by a maximum of less than 1 cfs, and filled all of the underground storage in approximately 1 month. Depending on the actual desired control conditions, it does not appear that the underground workings are capable of providing any meaningful storage for operations management on an annual basis.

6.0 Geotechnics

Seepage and stability analyses were conducted for select proposed SLT hydraulic control alternatives. Analyses were performed for the debris plug and adjacent and overlying colluvium as discussed in Section 6.1. The stability of slopes in the terrain trap and the slope immediately south of the open, collapsed reach of the SLT was also analyzed as presented in Section 6.2.

6.1 Debris plug/colluvium

Two geometric configurations were analyzed corresponding to the only slightly modified geometry of Alternatives 0 through 2 and the added fill geometry of Alternative 3.

The anticipated backup of water in the tunnel (upgradient of the existing debris plug) during high runoff events as described in Section 5.2 will result in increased pore pressures and elevated seepage gradients in the colluvium adjacent to and overlying the debris plug. The following potential failure modes resulting from these hydraulic loadings were analyzed: 1) excessive exit gradients that cause uplift (heave) in cohesionless soils; 2) uplift pressures (blowout) in cohesive soils; and 3) slope instability. It is important to note that the analyses performed herein assume that the existing casing at AT-2 and relief wells to be installed as key elements of Alternatives 2 and 3 are maintained to function as designed. The redundant relief wells planned for Alternatives 2 and 3 are not included in the analyses performed herein due in large part to the results of hydraulic modeling of Alternative 3 shown on Figure 5.3 that demonstrate that the additional value of more wells to control head build-up is limited.

Analysis of failure due to piping was specifically not analyzed; however, further development of alternatives (and final design if one of the alternatives analyzed herein is selected) would need to consider appropriate filter and drain design, or other mitigation, if piping failure is to be avoided.

6.1.1 Methodology

Two-dimensional finite element and limit equilibrium analyses utilizing the Seep/W and Slope/W software from Geoslope International were used to model seepage from the tunnel through colluvium and the debris plug, and slope stability, respectively. Exit gradients are output directly from Seep/W. For analysis of exit gradient factors of safety and uplift pressure factors of safety, the methodology provided in Design Standard No. 13 - Embankment Dams (USBR, 2011) was utilized. Design Standard No. 13 recommends the following factors of safety for analysis:

Recommended Factors of Safety Against Heave*

Type of Facility	Recommended Factor of Safety
New Dam	3.0
Existing Dam	4.0

*USBR guidance indicates that a lower Factor of Safety of 2.0 to 2.5 is acceptable if soil properties are well understood and a piezometer array is available to measure pressures.

Recommended Factors of Safety Against Uplift

Type of Facility	Recommended Factor of Safety
New Dam	2.0
Existing Dam	1.5

Although these criteria were developed for conventional water retention dams they are regarded as appropriate to the debris plug and adjacent colluvium as these materials function, in effect, as a dam storing the water backed-up in the SLT, and their failure could cause damage at the site and in an extreme event possible overtopping of the flood dike as discussed in Section 7.2.

6.1.2 Assumptions

Several key assumptions were necessary to simplify the analyses:

- The 3-D tunnel geometry is modeled via 2-D analyses, which is conservative for both seepage and slope stability.
- Permeability (i.e., hydraulic conductivity) of colluvial materials is estimated conservatively. Permeability of the debris plug is based on the tunnel flow calibration analyses presented in Section 5.2.
- Colluvial soils in the vicinity of the tunnel can be considered as having characteristics and properties between those typical for cohesionless and cohesive soils. For purposes of analysis, both heave (cohesionless soils) and uplift or blowout (cohesive soils) are checked.
- While tunnel heads are expected to be transient, steady-state conditions are conservatively assumed for these analyses. As seen on Figure 5.4, seasonally high flows for more extreme events can last for several months. More detailed examination of the simulated mine discharges shown at small scale on Figure 5.1 indicates that even more typical seasonally high heads can remain elevated for up to several weeks.

6.1.3 Results

Curves were developed for appropriate increments of tunnel head for the two geometries and the various analyses applicable to Alternatives 0 through 3. Factor of safety results for Alternatives 0 through 2 are presented on Figure 6.1. The output for seepage and stability analyses and the results of uplift factor of safety calculations for Alternatives 0 through 2 are presented on a series of figures in the first part of Appendix A, grouped by tunnel head.

These results indicate that a phreatic line builds up within the colluvium to nearly the elevation of the available tunnel head. For tunnel heads above 8870, a seepage face develops on the colluvial slope. The results of exit gradient, slope stability, and uplift analyses all indicate that factors of safety are not considered adequate for tunnel heads above 8870. On this basis, Alternative 1 does not meet the factor of safety criteria for the case of the head associated with the maximum modeled inflow over the 60-year model period. Exit gradient and uplift analyses at 8870 and below are not applicable and slope stability is controlled by the upper colluvial slope, as opposed to tunnel head and seepage. Slope stability factors of safety for tunnel heads lower than elevation 8870 are greater than 1.5 based on extrapolation of the relevant curve on Figure 6.1 and thus acceptable.

For Alternative 3, the concrete plug to be placed upgradient of the debris plug acts as a seepage barrier, and the debris plug is conservatively assumed to maintain its existing permeability and thus acts as a relatively permeable drain. Results indicate that the added permanent fill to be placed in the terrain trap under Alternative 3 prevents development of a seepage face for tunnel heads up to elevation 8892. At this tunnel head, uplift and exit gradient analyses are not applicable and slope stability is adequate at 1.5 and not controlled by tunnel head and seepage, but rather by the upper colluvial slope. The output files for seepage and stability analyses for Alternative 3 are presented on two figures at the end of the first part of Appendix A.

It should be noted that high gradients develop around the concrete plug for Alternative 3, and the adjacent colluvial soils are subject to high internal erosion forces. Risk of internal erosion from the colluvium into the existing debris plug should be considered to be high due to the high gradients and the likelihood that the debris plug in its current condition will be unable to provide adequate filtration. This condition could theoretically be mitigated but only if the debris plug was properly filtered, effectively blinded at the contact with the colluvium, or sufficiently plugged to very significantly reduce the inferred existing void space.

6.2 Terrain trap and adjacent slopes

In order to assess the stability of existing slopes in the terrain trap and open, collapsed tunnel area, a total of three representative profiles (i.e., cross-sections) were selected in the locations and orientations shown on Figure 6.2. For the purposes of the stability analyses, it was assumed based on the results of prior and ongoing field investigations that the subsurface profile in each of these areas consists entirely of silty sand, and sand and gravel colluvial materials. Based on laboratory tests of remolded samples of this material, lower bound strength parameters, including a very slight cohesion of 50 psf and an angle of internal friction of 38 degrees, were selected for use in the analyses. For each selected cross-section, three cases were evaluated including:

- Static conditions with dry soils
- A pseudo-static seismic (i.e., earthquake) loading of 0.027g, derived from horizontal ground accelerations consistent with an earthquake return interval of 1 in 475 years
- Surface soils wetted to a depth of 3 feet by snowmelt, or a heavy rainfall event. For this case, the saturated soils were assumed to exhibit no cohesion.

Slopes were evaluated for stability by means of the STABL5 computer program, employing the Modified Bishop method of analysis. The program was configured to assess both shallow and deep shear surfaces extending from several points at the base of the slopes to multiple points along the face and tops of the slopes.

The safety factors generated by these analyses are summarized in the table below:

CASE	PROFILE 1	PROFILE 2	PROFILE 3
Static – Dry	1.5	1.2	1.3
Earthquake	1.4	1.1	

* The lower safety factor represents very shallow surficial sloughing of the saturated soils, the higher factor of safety represents slides extending to greater depth.

In general, a factor of safety of 1.5 is considered desirable for long-term stability of slopes for static conditions. For short-term conditions, such as during limited periods of construction, a reduced factor of safety of 1.2 is judged acceptable in this case. A factor of safety of 1.1 is acceptable for seismic events when calculated using the inherently conservative pseudo-static method. As shown in the table above, these criteria for the stability of slopes is met for each of the profiles assessed as part of this study for the short-term (i.e., construction period) exposures, except for very shallow sloughing associated with saturation of the surface soils.

In addition to the above analyses, sensitivity analyses were performed using very conservative lower-bound strength values based upon extensive experience with comparable natural non-cohesive soils. For this alternative analysis, an angle of internal friction of 35 degrees was assumed, with no cohesion. These analyses yielded the following factors of safety:

CASE	PROFILE 1	PROFILE 2	PROFILE 3
Static- Dry	1.2	1.0	1.0
Earthquake	1.1	1.0	1.0
Saturated Surface Soils	0.6/1.2	0.8/1.0	0.6/1.0

The stability analyses output files for all 18 cases are provided in the second part of Appendix A.

7.0 Risk Assessment

Implementation (including both construction and operation) of each of the alternatives described in Section 4.0 involves risks. The types of risks identified and evaluated include:

- Debris plug failure-related risks (Alternatives 1 through 3 only)
  - High inflows and/or plugging of the existing debris plug in the colluvial section of the SLT
  - Resulting increase in head and backed up water in the SLT
  - Rapid or long-term failure of the debris plug and/or the adjacent and overlying colluvium
  - Uncontrolled release of tunnel water, metals-bearing solids and fine-grained soils to the Ponds System and possibly the Dolores River
- Change in the point of surface or near-surface discharge of SLT flows from the debris plug (Alternatives 1 through 3 only)
- Development of seeps on the slopes above and adjacent to the SLT resulting from plugging of the debris plug and increased head in the tunnel with accumulation of mine water (Alternatives 1 through 3 only)
- Instability of the slopes above and adjacent to the SLT resulting from plugging of the debris plug (Alternatives 1 through 3 only)
- Infiltration of SLT dissolved metals-bearing discharge flows to colluvium, then to alluvium, and thereby to the Dolores River (Alternatives 1 through 3 only)
- Refusal in cobble to boulder colluvium prior to reaching drilling targets resulting in multiple attempts, especially with angled borings
- Challenges in placing a tunnel plug remotely through a boring and achieving an adequate hydraulic seal, with potential for a rapid or long-term failure of the plug by loss of attachment and displacement due to water pressure, and/or erosion of the tunnel wall due to locally higher velocity flow
- Rockfall and/or debris slides while accessing and working within the terrain trap due to natural causes (e.g., near surface saturation due to heavy rainfall or rapid snow melt; earthquake shaking) or ground disturbance during construction or operation (e.g., vibrations from drilling, pipe ramming and other construction equipment; excavations on or near the slopes)
- Working with and around heavy equipment including drill rigs, cranes and earthmoving equipment
- Working near water under pressure within the existing St. Louis Tunnel

These risks are each discussed and qualitatively evaluated below.

7.1 Geotechnical risks

7.1.1 Debris plug/colluvium

The existing debris plug backs up water in the SLT as documented by measurements of the head in the casing of AT-2 which is interpreted as intercepting an open portion of the tunnel upgradient of the inferred debris plug. Hydraulic modeling as discussed in Section 5.0 demonstrates that even without any reduction of the current hydraulic conductivity of the plug, head will build up when inflows from the extensive workings upgradient increase seasonally or during years with higher precipitation. Thus, it was necessary to assess a full range of potential hydraulic conditions (Section 5.0) and geotechnical loadings resulting from those hydraulic conditions (Section 6.0). Based on those analyses, the potential for and consequences of failure of the debris plug and/or the adjacent and overlying colluvium under anticipated and potentially more severe future inflows are evaluated (with and without plugging or blinding of the debris plug). The potential consequences of a geotechnical failure are discussed in Section 7.2.

No plugging. As discussed in Section 6.0, the maximum hydraulic heads within the SLT due to modeled mine inflows for the past 60-year period for Alternative 1 (with only AT-2 acting as a relief well) result in unacceptable factors of safety for all three potential geotechnical failure modes analyzed. By extrapolation, Alternative 1 would clearly fail to meet the geotechnical criteria for the heads associated with any of the more severe recurrence interval mine inflows. Alternative 1 does, however, meet the geotechnical criteria for the average modeled head.

Assuming two fully functional 6- or 8-inch ID relief wells (and with no reliance on one or more redundant wells), all of the geotechnical Factors of Safety are met for Alternative 2 for all modeled flows up to the head associated with the very conservatively estimated 100-year flow. As shown on Figure 7.1, the maximum heads modeled for the estimated 100-year peak discharge event are readily accommodated by two 8-inch relief wells (Alternative 2b) and with less safety by two 6-inch relief wells (Alternative 2a).

Alternative 3 meets all three of the geotechnical factor of safety criteria for the maximum head over the 60-year modeled period regardless of the number of 6-inch relief wells assumed. As noted previously in Section 6.0, this is due to the dissipation of head in the permanent fill to be placed over the colluvium in the terrain trap under this alternative. Although not specifically analyzed, it appears that Alternative 3 may fail to meet the geotechnical criteria for the most conservative heads associated with the 100- and possibly the 50-year recurrence mine inflows, but may still meet criteria for the 25-year flow. Additional analyses would be performed if Alternative 3 is selected for further consideration.

Plugging. The debris plug is judged susceptible to sudden or long-term reduction in hydraulic conductivity (permeability). Such reduction could result, for example, from seismic shaking leading to collapse of highly conductive conduits formed by a metastable arrangement of broken timber supports and gravel, cobbles and boulders that may be present. Long-term reduction of hydraulic conductivity could occur due to precipitation of primarily iron oxides, hydroxides and/or oxyhydroxides (i.e., “red dog” or “yellow dog”) on and around the soil/rock assumed comprising much of the plug that eventually accumulates to the point of partially to completely filling the voids in the debris plug.

The probability of either sudden or long-term significant reduction of the hydraulic conductivity (i.e., “plugging” or “blinding”) of the debris plug is unknown and not amenable to quantitative determination. Based on the available hydraulic and geotechnical data and relevant experience, the probability of either event is judged relatively low but not negligible. If such plugging was to occur it would result in further back-up of water in the SLT (and possibly the cross-cuts and interconnected higher workings) beyond that already predicted to accompany occasional high inflows as modeled for the past 60-year period.

The potential effects on debris plug and/or colluvium geotechnical behavior resulting from increased head due to reduction of debris plug hydraulic conductivity from long-term blinding or sudden collapse can be inferred based on the analyses summarized in Table 5.4. The results for Alternative 1 are apparent as that alternative failed to meet any of the geotechnical criteria even with the debris plug at its current assumed relatively high bulk hydraulic conductivity. To the contrary, it is apparent that for Alternative 2 with only two 6-inch relief wells (and no reliance on one or more redundant wells) that the geotechnical criteria would be met even if very substantial reduction in hydraulic conductivity occurs (up to at least four orders of magnitude). Of course literally complete blinding to effectively zero hydraulic conductivity would result in very rapid and severe increase in head, but such a condition is very unlikely to occur and would be apparent from the monitoring proposed for all of the alternatives. Although not specifically analyzed, Alternative 3 is anticipated to behave somewhere between Alternatives 1 and 2, likely closer to Alternative 1.

Consequences. Based on the hydraulic and geotechnical analyses performed to date, it appears that the potential for failure of the debris plug and/or the adjacent and overlying colluvium is relatively high for Alternative 1, remote for Alternative 2, and low for Alternative 3. Given these potentials, it was determined both helpful to support judgments as to the degree of acceptable risk for Alternatives 1 and 2, and prudent for Alternative 3, to evaluate the potential consequences should a failure occur. This evaluation also provides a basis to understand the unlikely, but not impossible, scenario that an undetectable geotechnical flaw is present in the colluvium and/or debris plug that would result in behavior less favorable than modeled with the assumed conservative assumptions and parameters used for the analyses presented in Section 6.0.

The consequences of breaching or blow-out of the debris plug and/or colluvium with uncontrolled discharge of accumulated mine water and precipitated metals-bearing solids (i.e., sludges) would depend on: 1) the head and volume of water backed up in the SLT; 2) how quickly the failure occurred; and 3) the flow conditions and features in the flow path downgradient. The potential consequences of a debris plug/colluvium breach are discussed in Section 7.2.

7.1.2 Terrain trap/collapsed area

Stability analyses were performed of representative slopes within the terrain trap and the open, collapsed reach of the SLT as described in Section 6.2. These analyses demonstrate that the existing slopes should be adequately stable against large-scale, deep-seated failure for any given short-term period of exposure (including the times that construction is occurring within the areas analyzed). However, the potential for localized, shallow sloughing is judged high during and after periods of significant rainfall or snowmelt that may saturate the upper few feet of the colluvium. Protection of workers and equipment should be provided against such potential shallow failures, in addition to rolling rocks coming down the slopes.

The fact that factors of safety of at least 1.0 were achieved in all cases for what is regarded as a very conservative minimum shear strength (except again for the case of very shallow sloughing of assumed fully saturated surface soils) supports the conclusion that large-scale, deep-seated slope failures are not anticipated for the existing slopes in the terrain trap and open, collapsed reach of the SLT during construction. However, it is also evident that some of the slopes are only marginally stable over the long-term with factors of safety less than 1.5 for static conditions using the less conservative design-basis shear strengths. Thus, long-term exposures of workers or facilities within the terrain trap or adjacent to the open, collapsed reach of the SLT should be avoided unless adequate measures are implemented to support the slopes, or engineered protection of workers and facilities is provided. Note that evaluations to date indicate that stabilizing the slopes to a long-term condition achieving a factor of safety of at least 1.5 is not technically feasible given the site conditions. The feasibility of designing protection to withstand potential deeper, larger scale failures possible over the long-term will depend on the nature of the facilities and activities to be protected.

7.2 Breach modeling

Based on the results of the hydraulic modeling described in Section 5.0 above and summarized in Table 5.2, the existing debris plug inferred present in a portion of the colluvial reach of the SLT as described in Section 2.0 causes a back-up of approximately 2 acre-feet (ac-ft) of water in the SLT under average inflow conditions resulting in a head above the invert at the upgradient end of the debris plug of approximately 12 feet. The storage and head under a modeled 60-year period maximum inflow are approximately 6 ac-ft and 29 feet, respectively. These estimates assume that the debris plug continues to drain as at present and that the casing in AT-2 is present, fully open (i.e., the existing 2-inch PVC pipe is removed), and would function as a relief well during periods of higher inflows. If the casing in AT-2 is plugged such that the only discharge path is through the existing debris plug, then the maximum storage and head would be on the order of 13 ac-ft and 44 feet, respectively.

The results of the geotechnical analyses described in Section 6.0 above and summarized on Figure 6.1 indicate that applicable Factors of Safety against failure of the colluvium overlying and adjacent to the debris plug and the colluvial reach of the SLT are not adequate for heads in the tunnel above elevation 8870 and that failure due to slope instability becomes likely (Factor of Safety < 1) at tunnel heads above elevation 8880, and due to heave or uplift above elevation 8885-8890.

Given these hydraulic and geotechnical results, a decision was made to model hypothetical failure of the existing debris plug under an appropriate range of conditions and assess the consequences of failure downgradient on the St. Louis Ponds site and potentially to the Dolores River. These analyses are described in the following subsections.

7.2.1 Modeling technique

An unsteady flood inundation model using the computer modeling software HEC-RAS version 4.1 was developed in attempt to quantify the order of magnitude of potential flooding impacts to the portion of the St. Louis Ponds site downgradient of the existing debris plug in the collapsed portion of the SLT to and including Pond 18 due to a hypothetical failure of the debris plug and/or overlying colluvial material. This included developing a model geometry file and a flow file which incorporated a dam breach component to simulate a failure of the tunnel debris plug and overlying colluvium. The flooding area of interest starts upstream at the upgradient end of the debris plug where the theoretical failure is considered most likely to occur, and follows a path moving west downstream, where it reaches Pond 18 and turns south through the pond. The modeled flood reach then extends about halfway through Pond 15 representing the downstream model boundary. This boundary was chosen given that the flood flows and stage further downgradient through the Ponds system are assumed to attenuate due to available storage in the system. Due to model limits ending before the Pond 15 embankment, any estimates of flooding downstream of the Pond 18 embankment are imprecise and would be subject to change should the model limits be extended farther downstream for future analyses.

The basis of the geometry file for the HEC-RAS model were grid contours developed by AECI in 2011 that are geo-spatially referenced in GIS. The flood flow path and cross-section lines were created in ArcMap, then exported to HEC-RAS using the HEC-GeoRAS tool. To simulate a “debris plug and colluvium” failure, an earthen dam was added to the geometry file at the upgradient end of the inferred debris plug reach of the SLT. Model geometry upstream of the dam was modified to simulate a reservoir, equivalent to the volume of stored water in the tunnel and cross-cuts, such that during the hypothetical dam failure, water was not routed through the thousands of feet of underground workings, but was available immediately upstream of the dam. This represents a conservative assumption, with little to no attenuation of flows in the model before they are discharged downstream.

The dam was modeled at 23 feet high spanning from the floor of the SLT at an elevation of 8851 feet to ground surface at elevation 8874. A stage-capacity curve was developed for the SLT and the SE/NW cross cuts as described in Section 5.2 (Figure 5.2). The total available storage in these underground mine openings is approximately 13.8 ac-ft. This stage-capacity curve was proportionally compressed from 46 feet to 23 feet while maintaining all of the storage to represent the 23-foot tall dam created in HEC-RAS (Figure 7.2). The cross-section geometry behind the dam was modified to the extent practical to simulate this compressed capacity curve. Modifying the cross-sections consisted of a trial and error process in order to closely match the total volume of 13.9 ac-ft at the highest water surface elevation behind the dam, and the corresponding volumes associated with the lower elevations. A profile view of the modified cross-sections and the implemented dam geometry is shown in Figure 7.3.

The empirically-derived Froehlich equations for determining dam breach failure parameters were used to estimate a breach failure time. Two scenarios were considered in assessing the breach failure time. The first considered the breach height to be the full 46 feet of head. The second considered the breach height to be the compressed, 23-foot dam height modeled in HEC-RAS. When the breach height was set to 46 feet, the failure time ranged from 2.5 to 4 minutes. At a height of 23 feet, the failure time ranged from 5 to 7 minutes. For this analysis, the failure time used was 5 minutes because it is a mid-range number between the 23 and 46 foot scenarios and a conservatively short failure time when considering the full 46 feet of head. Note that the 23-foot high dam is a physically more realistic model of the colluvium over the debris plug, but that the head acting on that dam is better modeled by the 46-foot head (more than half of which is confined by the Hermosa formation bedrock and acts on the colluvium only through the SLT). The assumed 5-minute breach failure is chosen to best accommodate the actual site conditions. A maximum breach width equal to the width of the tunnel was set at 9 feet.

Several modifications were made to the model geometry to stabilize the model and maintain channelized flow in order to assess the potential flooding impacts. These included the following:

- Cross-sections were interpolated every 20 feet.
- Cross-sections 888 through 1222 were lowered linearly from the elevation of the dam, 8851 feet. The adjustments to the cross-sections were done in-between the bank stations only (Figure 7.3).
- Several cross-section lines were bent to end at a higher ground surface elevation where there was not a well-defined channel and sheet flow would most likely occur. This was done to maintain a channelized flow directing the flood through Pond 18.
- In addition to the bent cross-section lines, levees were implemented into the model along the flood dike and a few other cross-sections with shallow channels. These were considered to be locations where the flood depth would most likely exceed and overtop the flood dike and existing grade.
- Due to the steepness of the slope upstream of Pond 18, high Manning's "n" values were used to lower the Froude number and minimize the potential for large changes in the energy grade that would otherwise destabilize the model.

Once the geometry file was complete, the dam breach parameters were entered into the model plan, and an unsteady flow analysis was computed. A stage and flow hydrograph was created for every cross-section, and the maximum flood inundation limits were plotted on a plan map, shown on 7.4A.

7.2.2 Modeled scenarios

Three scenarios were modeled based on results from the hydraulics analysis for the simulated inflow data presented in Section 5.2; the specifics of each are described in the following paragraphs.

The first model scenario represents Alternative 0 or an extreme case assuming that no action is taken to mitigate the current conditions, and that the casing at AT-2 becomes plugged and non-functional from vandalism or failure to maintain the casing from scaling and eventual plugging. This scenario is not expected to occur, but is provided for perspective when compared to subsequent scenarios. For this case, the hydraulic analysis in Section 5.2 indicates a maximum head in the tunnel of 44 feet above the invert at the dam (immediately upstream of the debris plug), thereby utilizing most of the storage capacity of the tunnel and the cross-cuts. The corresponding stage and storage was determined from the compressed stage-storage curve presented in Figure 7.2.

The second model scenario represents a more probable case of failure during a maximum modeled inflow event, and assumes that the conditions match Alternative 1, with AT-2 acting as a relief well. This condition was selected to model the base case alternative, and provide perspective of potential consequences between the extreme Alternative 0 conditions and the lowest head conditions associated with Alternatives 2a/2b (the third scenario modeled). Under this scenario, 29 feet of head was assumed to have built up behind the 23-foot high dam modeled in HEC-RAS. This was input into the model by compressing the stage-storage curve for 29 feet of head, or 6.2 ac-ft of water, down to 15 feet as described above for the first scenario. No additional modifications were made to the model for this scenario.

The third model scenario represents hypothetical failure conditions associated with Alternative 2, assuming two six inch relief wells (with AT-2 capped), and a fully clogged debris plug. This scenario was modeled to represent a conservative failure condition assuming Alternative 2 is implemented, but that the debris plug is eventually blinded and no additional relief well capacity is provided. This again is an unlikely scenario, assuming that monitoring would detect the blinding and that additional relief capacity would be provided. Based on the hydraulic results under this scenario, 15 feet of head was assumed to have built up behind the 23-foot high dam modeled in HEC-RAS. This was input into the model by adjusting the stage-storage curve for 15 feet of head, or 2.8 ac-ft of water, down to 7 feet of head behind the 23 foot dam. For consistency with the methodology applied to the previous two model scenarios, the 15 feet of head was used to determine the corresponding head associated with the compressed stage-storage curve. No additional modifications were made to the model for this scenario.

7.2.3 Results

The peak flow rate from the dam break under the first scenario was approximately 1,824 cfs. Based on a review of the topography surrounding the model extent, there are multiple areas along the reach where the flood would likely be diverted to alternate flow paths and provide attenuation, thus reducing the peak flow rate at Pond 18. Therefore, the routed flows and flood stages based on a peak breach hydrograph flow of 1,824 cfs represent conservative estimates of the effects of a debris plug/colluvial failure under the aforementioned assumptions. The peak flow rate under the second scenario was 678 cfs and under the third scenario, 238 cfs.

Figure 7.4A provides a plan view of the maximum (conservative) flood inundation limits for the first scenario and identifies the cross-section numbering used to describe the model results. Cross-sections 1222.0 through 650.6 represent the flood reach from the end of the tunnel to the top of the Pond 18 storage area. Cross-sections 626.6 through 277.7 represent Pond 18, in which the right bank is the Dolores River flood dike. Cross-section 230 is the Pond 18 dam embankment, and cross-sections 155.9 through 29.0 represent the upper portion of Pond 15.

The dam break model results indicate that water would over-top the Dolores River flood dike between sections 29 and 472 for the first model scenario, between cross sections 65 and 341 under the second model scenario (Figures 7.4A and 7.4B), and between cross sections 103 and 131 under the third model scenario (Figure 7.4C). Upstream of Pond 18 for the first model scenario, the water would over-top the natural grade at sections 823.5 and 907.3. The flood dike at cross-section 131.1 was overtopped by approximately 2 feet under the first model scenario, representing the maximum overtopping depth for the reach modeled (Figure 7.5A). The same section was also overtopped in the second and third model scenarios, but only by approximately 1 and 0.5 feet of water, respectively (Figures 7.5B and 7.5C). Further upstream at cross-section 341, the flood dike was overtopped in the first model scenario by approximately 0.5 feet and was not overtopped in the second or third model scenarios (Figures 7.6A, 7.6B, and 7.6C). Figures 7.4B and 7.4C show that the upstream portion of the flood dike in Pond 15 (just below the Pond 18 embankment), was the only portion overtopped during the second and third model scenarios. Note that the contour elevations shown on Figures 7.4A, B and C are from 2011, and the overtopping depths at the flood dikes were calculated based on flood dike upgrades implemented after the 2011 survey.

All model scenarios suggest that the flood dike will be overtopped adjacent to Pond 15. However, there are a number of intentional conservatisms in the modeling as described above. If any of the alternatives depending on the debris plug/colluvium remaining in place are selected to advance beyond this level of study, it is recommended that more detailed hydraulic analyses be performed to better assess the degree of conservatism in the current modeling. If such additional modeling still indicates the potential need for downstream risk mitigation based on predicted hydraulic conditions associated with the selected design alternative, then one or more of the following potential mitigation tactics could be pursued:

- Further evaluate potential risks by refining the model inputs and extending the model domain downstream to Pond 5
- Raise the flood dike to prevent direct discharge to the Dolores River
- Raise upstream pond embankments to provide incremental additional flood storage and resulting attenuation of peak flow
- Provide an enhanced overflow spillway into Pond 10 to utilize available storage to further attenuate flow and provide for settling of entrained precipitated metal sludges and sediment prior to discharge through the remaining lower ponds and to the Dolores River

Note that if any mitigation for a debris plug/colluvium breach flood is envisioned, the evaluation and design of any such measures should be coordinated with decisions regarding the long-term use or fate of the lower ponds and whether protection of those ponds from flood flows on the Dolores River is necessary. Such studies are beyond the scope of this current effort.

7.3 Seepage losses

Current losses of SLT discharges by infiltration to the colluvium out-by the rock portion of the tunnel are not known but are believed to be relatively minor, and possibly within the precision with which the flows can be measured. The bases of this current belief are as discussed below. Ongoing field investigations as described in the Adit and Portal Investigation Report – 2013 Update (Atlantic Richfield Company, 2013) will provide substantial additional basis to evaluate the current losses. The results of updated estimates of the seepage loss will be presented in an addendum to this PDR upon completion of the field work and analyses.

7.3.1 Hydraulic conductivity of colluvium

The bulk hydraulic conductivity (permeability) of the colluvium is estimated based on available gradations to average in the range of 10^{-3}-10^{-4} cm/sec, and to be as low as 10^{-5} cm/sec in silty fine sand and as high as 10^{-1} cm/sec in relatively clean sandy gravel to gravelly sand. Using these assumed hydraulic conductivities, the area of the floor of the tunnel in the collapsed/colluvial reach of the original tunnel (330 ft x 9 ft = +/- 3000 sf), and a hydraulic gradient of 1 (conservatively assuming infiltration through the tunnel floor to an unsaturated interval of colluvium), infiltration rates are estimated by the Darcy equation (Q = kiA) as:

0.4 gpm (0.0009 cfs) for k = 10^{-5} cm/sec

4-44 gpm (0.009-0.09 cfs) for k = 10^{-4}-10^{-3} cm/sec

4400 gpm (9.8 cfs) for k = 10^{-1} cm/sec

Given the measured flows at DR-3 and the results of the Rico Site Underground Mine Workings Hydraulic Model discussed previously, the average assumed range of hydraulic conductivity and associated losses appear reasonable to well within plus or minus one order of magnitude. In other words, it seems plausible that losses are somewhere in the range of a few tens to a few hundreds of gallons per minute. Of course it is possible that there are some higher hydraulic conductivity channels within the colluvium that would result in locally greater infiltration, but not likely to the high end of the bulk hydraulic conductivity assumed above.

7.3.2 Groundwater levels

The available groundwater level data and boring logs for monitoring wells in the immediate vicinity of the collapsed/colluvial portion of the SLT suggest that there is shallow perched groundwater at and just above the invert of the SLT at approximately elevation 8850-8855 (CHV-101D, MW-204). Two other perched zones were encountered at approximately 15 and 20 feet below the invert at about elevation 8835 and 8830, respectively (CHV-101D). The ambient groundwater table in the colluvium is on the order of 30-35 feet below the tunnel invert at elevation 8815-8820 (CHV-101S, MW-202, MW-205). These data indicate some loss may be occurring laterally from the tunnel to the shallow perched zone, and do not rule out that some leakage is occurring to deeper perched aquifers and the water table. However, the seepage loss is at least not sufficient to form a groundwater mound from the ambient water table to the invert of the SLT as might be anticipated if losses were very high.

7.3.3 Geochemical data

Water samples have been collected from MW-204, CHV-101S, and AT-2 on several occasions between November 2012 and March 2013. In addition, a water sample was collected from BAH-01 in October 2012. Monthly water samples are collected from the SLT surface discharge (DR-3) and the monitoring wells on the valley floor. These samples have been analyzed for general minerals (cations and anions) and dissolved and total metals. The analytical results from these samples have been compared in several ways, including Stiff diagrams, Piper diagrams, and bar graphs for total and dissolved metals.

Groundwater samples from monitoring wells MW-204 and CHV-101S indicate similar general mineral character (calcium sulfate) as the SLT discharge at DR-3. The water from AT-2 (from the submerged portion of the tunnel above the debris plug) is also similar, although depleted in bicarbonate. The strong calcium sulfate character is indicative of water moving through a mineralized zone high in sulfides. It would generally contraindicate a source from the surrounding Hermosa Formation bedrock, which comprises sandstone, siltstone, and limestone. It would also contraindicate a source of meteoric water passing through talus and colluvium, which would be derived primarily from the Hermosa Formation. The surface water in the area, which passes through the Pennsylvanian formations north of the site, yields a calcium bicarbonate water of low TDS (as evidenced by the water quality at DR-1 and GW-1). The similarity in general mineral character between the monitoring wells adjacent to the reach of the SLT in colluvium and the water discharged at DR-3, indicates a similar source for both, which would be the tunnel.

Total metals data indicate a continuity of decreasing metals concentrations, moving from east to west along the colluvial portion of the tunnel alignment. That continuity likely results from infiltration of adit water into the colluvial materials along the tunnel alignment.

Seeps and springs issue from colluvium along the eastern edge of the Dolores River flood plain. These discharges indicate that some water is moving through the colluvium and likely mixing with the water discharging from the tunnel along the eastern boundary of the flood plain.

The geochemical data indicate some infiltration occurring from the SLT discharge into the colluvium beneath and immediately adjacent to the colluvial reach of the tunnel. However, existing groundwater monitoring wells are insufficient to quantify the amount of groundwater leaking from the tunnel and into the sediments beneath the valley floor. As noted previously, ongoing field investigations are providing the basis to estimate these losses at least to order-of-magnitude.

7.4 Construction

The construction activities associated with all six of the action alternatives are characterized by inherent risks due to the ground conditions, weather, type and numbers of equipment required, potential for simultaneous operations, and the nature and complexity of the tasks required to construct the alternatives. Atlantic Richfield Company implements a thorough, ongoing program of health, safety, security and environmental protection (HSSE) protocols and practices both during design and construction to mitigate these risks. As a result, any alternative(s) recommended for advancement from preliminary to final design will meet Atlantic Richfield Company’s HSSE requirements.

7.5 Operations and maintenance

Atlantic Richfield Company’s HSSE protocols and practices carry forward into operations and maintenance. Again, any alternative(s) recommended for advancement from preliminary to final design will meet Atlantic Richfield Company’s HSSE requirements.

8.0 Recommended Alternatives

The six action alternatives identified and characterized in Section 4.0 were individually evaluated and compared in terms of a set of selected key factors as discussed in Section 8.1. Section 8.2 presents the rationale for identifying a preferred alternative to carry forward to 30-percent design, and ultimately to final design and construction, based on the comparative evaluation.

8.1 Individual and comparative evaluations

Each of the six action alternatives were first evaluated individually in terms of the following key factors:

- **Meeting project objectives:** How well does the alternative achieve the project objectives? It is required that all of the alternatives meet at least a minimum threshold of compliance.
- **Technical feasibility:** Are there any apparent or potential fatal flaws or issues of significant concern that could prevent producing a constructible and functional design?
- **Constructibility:** Can the alternative be constructed without extraordinary means and methods given the known and potential constraints at the site (access, working space, safety, weather, etc.)? Can the work be completed within the project schedule completion date(s)?
- **Operational considerations:** Are there operational requirements that are non-routine, time consuming, complex, and/or uncertain? Are seasonal weather conditions a significant factor in operations? What are the staffing requirements?
- **Monitoring requirements:** What is the scope, frequency and complexity of monitoring required? Can monitoring be accomplished remotely? Are there unusual or challenging conditions to overcome?
- **Relative risk:** What are the significant remaining relative risks during final design, construction, and operation and maintenance that are not fully mitigated by the proposed alternative?

As for Section 4.0, the following individual evaluations of each alternative will build upon the preceding alternative where appropriate, with only additional or significantly different conditions or issues discussed.

8.1.1 Alternative 1 – Base case

The base case was intentionally identified and characterized to represent the existing conditions within the terrain trap. No immediate measures are proposed to provide for periodic seasonal or wet-year increased hydraulic heads and accompanying water accumulation in the SLT. Potential sudden or long-term changes to the hydraulic or geotechnical characteristics of the existing debris plug are not addressed by up-front measures, but rather by proposed monitoring and subsequent response if/as required. Potential seepage losses, if determined significant by ongoing studies, are addressed only downgradient of the debris plug.

Alternative 1 is technically feasible and constructible. The primary technical challenge is to design shoring and a sequence of construction to maintain the stability of the south slope immediately adjacent to the open, collapsed reach of the SLT during debris removal and conveyance construction. Other design challenges include providing for continued conveyance of mine water discharges to treatment during the winter and minimizing to the extent feasible blockages of the conveyance by beavers during long-term operations.

The proposed remedy under Alternative 1 will operate by gravity drainage of the mine water as at present. No permanently installed mechanical or powered equipment or facilities are required. Periodic removal of debris (e.g., locally settled solids, wind-deposited sediment, organic debris including beaver dams) from the new mine water conveyance channel/pipe will be required, but is not anticipated to be frequent or especially burdensome. Full-time staffing will not be required for ongoing operations and maintenance. It is anticipated that minor maintenance of the facilities may be required on a biannual to at most quarterly basis.

The locations, types and approximate frequency of monitoring for Alternative 1 are as described in Section 4.1.3. The most critical monitoring will be for the hydraulic head in the SLT above the debris plug, and the accompanying monitoring of flow below the debris plug. These data will provide the basis to assess if the hydraulic conductivity of the debris plug is has decreased suddenly or is decreasing progressively over time. The primary challenge for this monitoring in the field will be maintaining the system components to mitigate such factors as scaling of instruments (e.g., pressure transducers monitoring hydraulic head in the tunnel), blockages in the conveyance system (as noted above), equipment/instrument failure (e.g., lightening damage). It will be critical that the monitoring data is both collected routinely and reviewed at appropriate intervals by qualified professionals to recognize if changes to the function of the debris plug are occurring so that timely action can be implemented if/as necessary.

Risk during design of the permanent work for Alternative 1 is judged minimal as the necessary data for required analyses is or will soon be available, and the analyses and designs envisioned are well within the bounds of established practice. There is some risk of triggering local failure of the slope south of the open, collapsed reach of the SLT during construction. However, this risk is judged manageable by proper design and strict adherence to the requirements regarding sequence, equipment type, and equipment location and loadings that will be specified for construction. Access to the terrain trap proper will only be required for short periods of time by personnel and possibly a drill rig to install monitoring instrumentation into AT-2 and/or a new monitoring well. The risk of rolling rocks and/or debris slides will be present but manageable by the means and measures described previously in Section 4.2.2. The primary risk associated with Alternative 1 is the sudden or long-term reduction of hydraulic conductivity of the debris plug and the resulting increase in hydraulic head and accumulated water within the tunnel. As discussed in Sections 7.1 and 7.2, there is a potential for geotechnical failure and uncontrolled release of accumulated mine water if monitoring fails to recognize sudden or long-term significant reduction of debris plug hydraulic conductivity, or if when recognized, timely response is not implemented. Assuming monitoring and any necessary response are timely and properly implemented, the risk of debris plug/colluvium failure is judged low.

8.1.2 Alternative 2 – Base case plus relief wells

The primary difference between Alternative 2 and Alternative 1 is that relief wells are pre-installed and designed to function without operator intervention should hydraulic head and accumulated water in the tunnel increase to levels above the design invert of the relief well system. The wells are designed to function under the natural head in the tunnel; no pumps, valves or other mechanical or electrical equipment is required. Potential seepage losses, if present, are dealt with in the same location and manner as for Alternative 1.

Alternative 2 is technically feasible and constructible. The major challenge for design and construction, in addition to those common to Alternative 1, is installing the two angled relief wells, possibly an additional redundant well, required piping, a manhole, and the conveyance culvert in the very limited work area well inside the terrain trap. The temporary excavation and work within that excavation required to plumb the relief wells into the manhole at the required invert elevation and with the necessary geometry to allow for periodic inspection and cleaning of scale will be challenging. Thorough design and construction planning will be required to ensure that this work can be done safely. Design of temporary excavation support will be critical, whether by Atlantic Richfield Company's or the contractor's fully qualified consultant.

Operations will be similar to those required for Alternative 1, except that periodic access into the terrain trap will be required to inspect the relief well piping, and clean scale as needed. These activities will occur in a confined space (i.e., a manhole) requiring strict adherence to relevant safety and health requirements. Conveyance channel maintenance outside of the terrain trap will be as for Alternative 1.

Monitoring will be as for Alternative 1. The opportunity will be available to provide redundant instrument installation, with two and possible three new wells to be installed, if determined desirable or necessary to accommodate the site conditions. Also, AT-2 could be abandoned if maintaining it free of scale is judged problematic or burdensome (as it was not designed as a permanent installation but rather to gather data to support design of the remedy).

The risks associated with Alternative 2 during construction are greater than for Alternative 1 given the longer duration required to complete the required work within the terrain trap, and the nature of that work including the required temporary supported excavation and installation of the relief well system components below grade. It is likely that medium to large equipment will be required to make and install support for the excavation (possibly involving vibratory sheet piling), and to set the manhole. Some hot work and/or cutting/grinding will be required to plumb the components. Although greater than for Alternative 1, these construction risks are manageable. Risk of geotechnical failure of the debris plug/colluvium during operation of Alternative 2 will be significantly less than for Alternative 1 given the relief well system. This system will be sized and located to effectively control even very infrequent large potential mine water inflows. Monitoring (and in this case routine inspection) is important to identify if scaling of the relief well system is occurring so that maintenance is performed as needed. However, the hydraulic design of Alternative 2 provides a very substantial reserve of capacity to accommodate high mine inflows as compared to Alternative 1. As a result, the risk of geotechnical failure of the debris plug/colluvium and uncontrolled release of accumulated mine water, although not negligible, is very much less than for Alternative 1 (again assuming that the system is properly and timely monitored and maintained).

8.1.3 Alternative 3 – Relief wells plus plugging tunnel

Alternative 3 is similar to Alternative 2 in terms of meeting project objectives in that relief wells are installed and designed to function without operator intervention. Potential seepage losses are again similar under this alternative as for Alternatives 1 and 2.

Although most of the components envisioned for Alternative 3 are judged technically feasible and constructible, there are two potentially problematic issues. As discussed in Sections 4.3.1 and 6.0, the installation of an effectively impermeable concrete plug upgradient of the existing debris plug may have the unintended negative consequence of inducing internal erosion (piping) of fines and sand from the colluvium into the porous debris plug. If this potential is not mitigated thoroughly by proper design and successful construction, the potential for a geotechnical failure resulting in uncontrolled release of accumulated water in the tunnel above the new concrete plug cannot be discounted. As noted in Section 4.3.1, the successful mitigation of this potential is uncertain given the challenging site conditions. The second issue is the potential for disturbance of the material within the debris plug during placement of the required permanent fill within the terrain trap resulting in unintended changes (possibly reduction) of the hydraulic conductivity of the debris plug. Again, as discussed in Section 4.3.1, mitigation of this potential is possible but not necessarily certain without extraordinary measures.

8.1.4 Alternative 4 – Interception wall

As discussed in Section 4.4.1, it was determined during evaluation of Alternative 4 that the concept was likely fatally flawed. This is due to the requirement that the colluvium enclosed within the interception wall be sufficiently permeable to transmit mine water inflows from the rock portion of the tunnel upward through the colluvium and overlying permanent fill to discharge at the surface of the fill pad. Based on the hydraulic analysis discussed in Section 5.2.2, it is apparent that the required hydraulic conductivity of the colluvium is many orders of magnitude greater than judged possible in order to avoid increase in head within the tunnel and back-up of mine water all the way to the Blaine (100) level.

8.1.5 Alternative 5 – Tunneling

Alternative 5 will fully meet the objectives of collecting all of the mine water discharge within the rock portion of the SLT and conveying it without measurable seepage loss to treatment. The existing debris plug will be removed as a component of the mine water conveyance by the installation of a secure concrete plug within the rock portion of the SLT that will divert mine water discharges to the rock tunnel/steel casing safely out of the terrain trap, and then to treatment via a new concrete lined channel (or RCP pipe). As a result, any future changes to the debris plug will have no effect on the mine water discharge. The open, collapsed portion of the SLT below the debris plug will be decommissioned by selectively removing some of the debris in the upper portion of the existing “channel” and backfilling with sand and gravel to mitigate any safety issues with unauthorized access to the area.

Alternative 5 is technically feasible and constructible. As described in Section 4.5.1, the proposed installation of large diameter steel casing by hydraulic pipe ramming through the colluvium and conventional rock tunneling to the intersection with the existing SLT are fully within the capability of appropriately experienced and equipped specialty geotechnical contractors. This is not to say that the construction is not without challenges given the site conditions. The primary construction issues anticipated are: 1) stabilization of the relatively loose colluvium ahead of the advancing face of the pipe ramming operation to prevent a running ground condition; 2) dealing with the potential to encounter some very large rock blocks within the colluvium during the pipe ramming; and 3) managing the water and solids accumulated within the SLT during breakthrough of the new tunneled reach into the existing tunnel. Each of these issues are encountered on many trenchless pipe installations and tunneling in mixed ground conditions, and as discussed in Section 4.5.1, each of these issues can be dealt with through proper design and employing appropriate construction techniques. Other aspects of the construction involve earthwork grading and concrete channel (or RCP) installation in conditions that are judged less challenging than for the preceding alternatives. This is due to: 1) only minimal work required within the terrain trap (installation and maintenance of ground movement instrumentation, and possibly some grouting ahead of the pipe ramming if safe and more efficient than grouting from within the bore); and 2) the routing of the new conveyance from the new “portal” of the 96-inch steel casing to the original SLT portal location being safely away from the steep slope adjacent to the open, collapsed reach of the tunnel.

Once constructed, there will be essentially no required operation or maintenance of the new casing/tunnel conveyance. Mine water discharges will flow at very shallow depth on the floor of the tunneled reach in rock and on the invert of the large diameter steel casing reach. Even under the most extreme, conservative flows estimated for a 100-year recurrence event the approximate depth of flow in this reach will be only on the order of 0.6 feet as compared to the 7-8 foot height of the tunnel and casing, respectively.

Given that it will not be possible for hydraulic head to increase and water to accumulate as at present in the new casing/tunnel reach or the existing SLT, monitoring of head in the tunnel is not required. However, it may be prudent to install a transducer at a weir installed at an accessible location in the new conveyance as a back-up to flow measurement downgradient at an improved DR-3 monitoring location. These redundant head/flow measurements would give warning if unanticipated but possible blockage somewhere back in the deeper mine workings were to occur due to overbreak in the tunnel back or walls and accumulation over time of precipitates as is occurring now behind the debris plug.

There are significant inherent risks associated with the pipe ramming and rock tunneling operations proposed for this alternative. These include: 1) the very large hydraulic pressures in the ram and the extreme forces applied by the hydraulic ram in driving the casing; 2) the potential for running ground at the face of the pipe ram during grouting operations; 3) potential for workers to need to enter the casing to clear a large rock block at times during the pipe ramming phase; 4) work underground in the rock tunneling, breakthrough, and concrete tunnel plug installation; and 5) work downgradient of the approximately 10-15 feet of head and accumulated water and solids in the existing SLT at the time of breakthrough. Although significant, these risks will be managed with thorough planning and implementation of all appropriate safety measures and practices during construction. On the other hand, once construction is complete there is essentially no remaining risk of debris plug/colluvium failure during the operational life of the system, and no need to enter the terrain trap and be subject to the potential for slope instability or rolling rocks.

8.1.6 Alternative 6 – Retaining wall

Like Alternative 5, Alternative 6 will fully meet the objectives of the project, and the existing debris plug will be removed as an issue as a result of the proposed construction.

Alternative 6 is technically feasible and constructible. However, as for Alternative 5, the construction means and methods require that the design be performed by appropriately qualified and experienced geotechnical and structural engineers, and that construction be performed by an equally qualified and experienced specialty geotechnical contractor(s). Given the depth from the required working fill pad to the SLT invert in the rock section of the tunnel, a very deep excavation retained by commensurately high permanent strut and waler supported micropile walls are required. While these techniques are commonly employed for a wide variety of civil/geotechnical projects in site conditions equally or more challenging than anticipated at Rico, they require very thorough geotechnical and structural design and construction with specialty equipment and techniques. Given the maximum height of the permanent walls involved to provide for at tunnel grade access to the rock portion of the SLT and the consequences of a failure, it would be critical that the design and construction are performed properly.

Again as for Alternative 5, no operation of the system will be required once it is in place and maintenance will be minimal during the anticipated design life of the structure.

Monitoring requirements are minimal with this alternative for the same reasons as discussed for Alternative 5.

Similar to Alternative 5, there are significant inherent risks during construction that would have to be thoroughly and effectively managed. These include: 1) potential disturbance of the existing colluvial portion of the SLT and the debris plug if the very substantial loadings imposed by the proposed flow fill and earth fill in the terrain trap are not fully and successfully accommodated during design and construction; 2) winching equipment and materials up a steep ramp to the top of the working platform, and or delivering same using a very large crane; 3) operation of high-energy percussion drilling equipment during micropile installation; placing of large steel walers and struts during excavation of the access corridor; 4) managing the existing water and solids in the SLT during breakthrough from the access corridor; and 5) working below the very high walls retaining the access corridor vertical slopes during breakthrough and installation of conveyance through the corridor reach. As for Alternative 5, there is no remaining risk of debris plug/colluvium failure once construction is complete. Although judged manageable with proper design and construction, there is greater risk during construction and later during operations for a local to global failure of the high retaining walls than for any kind of failure of the casing/tunnel under Alternative 5.

8.2 Recommended alternative

Alternative 1 only meets the project objectives in the most limited sense, and the risk of debris plug/colluvial failure, although not high, is judged higher for this alternative than for any other of the five alternatives. It is recommended that Alternative 1 not be pursued further.

It is recommended that Alternative 3 be set aside and not pursued further given: 1) the problematic geotechnical issues noted above; 2) the fact that time spent within the terrain trap and the work required will expose workers and equipment to even higher construction period risks than for Alternative 2; and 3) that the hydraulic head under this alternative will be greater than for Alternative 2, with reliance on the integrity of the overlying permanent fill to mitigate potential geotechnical failure.

Based on the apparent fatal design flaw as described in Section 4.4.1 and noted above, it is recommended that Alternative 4 not be pursued further.

Given the anticipated greater construction period and long-term risks associated with the high retaining walls required as described in Section 4.6.1 and noted above, and in the absence of any apparent significant benefits or advantages not also present with Alternative 5, it is recommended that Alternative 6 be set aside and not pursued further.

Alternative 2 is recommended to advance to final design for the following reasons: 1) this alternative reasonably meets the project objectives, assuming that the preliminary conclusion that seepage losses from the colluvial portion of the SLT and debris plug are not large is verified by ongoing field investigations and subsequent analyses; 2) the project design provides very substantial reserve capacity to accommodate even very high mine water inflows; 3) although work must occur in a very limited, and for some tasks, confined space, the scope and nature of the work required are relatively routine and can be performed safely; 4) the planned monitoring will provide ample warning of sudden or long-term reduction of hydraulic conductivity of the debris plug and allow timely and effective response to mitigate such a condition; and 5) implementation of this alternative does not preclude implementation of Alternative 5 at a later date if the performance of Alternative 2 is found inadequate at any point in the future.

It is proposed to retain Alternative 5 as a potential future replacement for Alternative 2 in the event that a condition or issue arises during operation and monitoring that brings into question the long-term adequacy of Alternative 2.

Given the above recommendations, the concepts described in Section 4.0 above for both Alternatives 2 and 5 have been advanced to the 30-percent design level as described in Section 9.0.

9.0 30-Percent Design

The concepts for Alternatives 2 and 5 as described in Section 4.2 and 4.5 and illustrated on Exhibits 4.2 and 4.5, respectively, have been further advanced to the preliminary (30-percent) design level. The primary objective of these preliminary designs is to assess the technical feasibility and constructibility of the key elements and features of the alternatives. At this level of design there is substantial flexibility intentionally retained for various elements of the designs, but the primary features, overall layout, and scale of the features are established.

The preliminary designs are presented on a series of Drawing Sheets for each alternative as listed in the Table of Contents. These include a cover sheet, overall plan layout, plan and profile sheets, and one or two section and detail sheets.

10.0 References

Atlantic Richfield Company. 2013. *Adit and Portal Investigation Report – 2013 Update, St. Louis Tunnel Hydraulic Control Measures, Rico Argentine Mine Site, Rico Tunnels Operable Unit OU01, Rico, Colorado*. October 30.

Interagency Advisory Committee on Water Data, 1982, *Guidelines for determining flood flow frequency: Bulletin 17B of the Hydrology Subcommittee, U.S. Geological Survey, Office of Water Data Coordination, Reston, VA,* 183 p

U.S. Bureau of Reclamation. 2011. D*esign Standards No. 13, Embankment Dams*. October.

U.S. Environmental Protection Agency (EPA). 2011a. *Unilateral Administrative Order for Removal Action, U.S. EPA Region 8, Docket No. CERCLA-08-2011-0005; Rico-Argentine Site, Dolores County, Colorado*. March 23.

U.S. Environmental Protection Agency (EPA). 2011b. *Removal Action Work Plan. Rico-Argentine Mine Site – Rico Tunnels Operable Unit OU01, Rico, Colorado*. March 9.

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Tables

Table 1.1: Summary of Alternatives

Alternatives	Components
1. Base case	• Drainage through debris plug • Monitor head thru AT-2 in tunnel to evaluate system hydraulics • AT-2 will function as relief well if triggered by elevated head in tunnel • Wing wall to intercept shallow leakage from debris plug and colluvial reach of tunnel and discharge to new conveyance downgradient of debris plug • Removal of existing debris in open, collapsed tunnel reach • Improve existing open, collapsed tunnel reach as channel with concrete lining or buried culvert pipe • Provide new concrete-lined channel to treatment
2. Base case plus relief wells	• Drainage through debris plug • Monitor head in tunnel thru AT-2 to evaluate system hydraulics • Inclined relief wells upgradient of debris plug; below grade tap to manhole to control head increase in tunnel • Drainage culvert to convey relief well discharges as gravity flow to new conveyance downgradient of debris plug • Wing wall to intercept shallow leakage from debris plug and colluvial reach of tunnel and discharge to new conveyance downgradient of debris plug • Removal of existing debris in open, collapsed tunnel reach • Improve existing open, collapsed tunnel reach as channel with concrete lining or buried culvert pipe • Provide new concrete-lined channel to treatment
3. Relief wells plus plugging tunnel	• Drainage through debris plug during construction, plugged after construction • Drilling platform fill; leave as permanent stabilizing buttress fill • Vertical relief wells from fill platform upgradient of new tunnel plug; below grade tap to manhole to control head increase in tunnel • Use vertical redundant vertical well for monitoring head in the tunnel • Plug tunnel at upgradient end of debris plug, downgradient of relief wells • Wing wall to intercept shallow leakage from debris plug and colluvial reach of tunnel and discharge to new conveyance downgradient of debris plug • Removal of existing debris in open, collapsed tunnel reach • Improve existing open, collapsed tunnel reach as channel with concrete lining or buried culvert pipe • Provide new concrete-lined channel to treatment

Table 1.1: Summary of Alternatives

Alternatives	Components
4. **Interception wall**	• Interception wall keyed into bedrock to minimize loss of tunnel drainage water to colluvium/alluvium upgradient of debris plug; secant piers drilled from a temporary elevated earthen fill platform to form continuous concrete wall • Water flow vertically up through colluvium to overflow thru a weir into a chute to new conveyance downgradient of debris plug • Monitoring well through colluvium and bedrock into downgradient reach of the rock portion of the tunnel • Removal of existing debris in open, collapsed tunnel reach • Improve existing open, collapsed tunnel reach as channel with concrete lining or buried culvert pipe • Provide new concrete-lined channel to treatment
5. **Tunnelling**	• Grading to construct launch pit and staging pad • Pipe ramming through colluvium with hand-mining as needed; rock tunnelling to breakthrough into existing St. Louis Tunnel in intact rock reach • Concrete plug in existing tunnel to divert all mine discharge to new casing/tunnel • Steel casing as liner in colluvial section; steel casing or conventional support in rock section • Steel grate gate at entrance to new tunnel casing; elevated steel grate floor to facilitate access to St. Louis Tunnel if desired and safe • Selective removal of shallow debris and earth backfill for existing open, collapsed portion of tunnel • New conveyance from portal of new casing/tunnel as channel with concrete lining or buried culvert pipe • Provide new concrete-lined channel to treatment
6. **Retaining wall**	• Elevated construction work platform using site fill; MSE wall to minimize fill encroachment on Soil Lead Repository • Micropile retaining walls (end wall and flanking tapered walls in corridor between Soil Lead Repository and open, collapsed reach of tunnel) supported by walers/struts to develop 25-foot wide 90-foot high retaining wall at competent rock upgradient of the colluvium/rock contact • Tapered flanking walls that extend 200 feet downgradient to provide at-grade access to new portal • Removal of existing debris in open, collapsed tunnel reach • New conveyance as concrete lining or buried culvert pipe from new portal through prior open, collapsed tunnel reach • Provide new concrete-lined channel to treatment

Table 5.1 Design Parameters Used in Hydraulic Model for Alternatives 1 through 4.

Design Alternative	Number of Relief Wells --	Relief Well Length (ft)	Relief Well Diam. (in)	Tunnel Invert Elev. (ft)	Relief Well Outlet Elevation (TOC) (ft)	Relief Well Angle (deg)	Debris Plug Flow? --	Bulk k-value (cm/s)	Sweep Angle (deg)	Sweep Radius (ft)
Alternative 0	0	--	--	8,851	--	--	Yes	3.9	--	--
Alternative 1	1 (AT-2)	21	4.25	8,851	8,866	32	Yes	3.9	--	--
Alternative 2a	2	13	6	8,851	8,861	32	Yes	3.9	45	5
Alternative 2b	2	13	8	8,851	8,861	32	Yes	3.9	45	5
Alternative 3	1,2,and 3	16	6	8,851	8,870	90	No	--	90	3
Alternative 4	0	--	--	8,851	--	--	No*	10^{-3}	--	--

Table 5.2 Output Statistics for Design Alternatives 0 through 3

Design Alternative	Max Head (ft)	Post 1999 Max Head (ft)	Average Head (ft)	Max Outflow (cfs)	Average Outflow (cfs)	Max Storage (cf)	Average Storage (cf)
Alternative 0	44	41	13	5.1	1.5	566,195	106,907
Alternative 1	29	27	12	5.2	1.5	272,276	89,660
Alternative 2a	13	12	9	5.2	1.5	96,864	60,104
Alternative 2b	11	11	9	5.2	1.5	78,281	58,946
Alternative 3 - One Well	42	39	22	5.1	1.5	466,748	197,515
Alternative 3 - Two Wells	25	24	20	5.2	1.5	232,016	174,611
Alternative 3 - Three Wells	22	21	19	5.2	1.5	196,295	170,360

Table 5.3 Alternative 4 Rating Curves for Range of k-Values.

	Alternative 4	
	$k=10^{-3}$ cm/s	$k=10^{-2}$ cm/s
Head (ft)	Flow (cfs)	Flow (cfs)
0	0	0
50	0.01	0.09
100	0.03	0.25
150	0.04	0.42
200	0.06	0.58
400	0.12	1.24
500	0.16	1.56

Table 5.4 Model Output Statistics for Variable k-Values for Design Alternatives 0 through 2.

Design Alternative	Hydraulic Conductivity (cm/s)	Max Head (ft)	Post 1999 Max Head (ft)	Average Head (ft)	Max Outflow (cfs)	Average Outflow (cfs)	Max Storage (cf)	Average Storage (cf)
Alternative 0	3.9	44	41	13	5.1	1.5	566,195	106,907
Alternative 1	3.9	29	27	12	5.2	1.5	272,276	89,660
	3	35	32	14	5.2	1.5	292,308	111,728
	2	43	40	17	5.1	1.5	515,469	145,029
Alternative 2	3.9	13	12	9	5.2	1.5	96,864	60,104
	3	13	12	7	5.2	1.5	101,235	66,525
	1	14	13	10	5.2	1.5	113,328	74,574
	0.1	15	14	11	5.2	1.5	120,230	76,056
	10^{-3}	15	14	11	5.2	1.5	121,068	76,238

Table 5.5 Log-Pearson III Probability Analysis Results for Simulated SLT Adit Flows

Recurrence Interval	Peak Flow Rate (cfs)
2	2.9
5	4.3
10	5.4
25	6.9
50	8.1
100	9.4
200	10.7

Table 5.6 Max Head Results for Log-Pearson III Peak Discharge Events at the SLT Adit

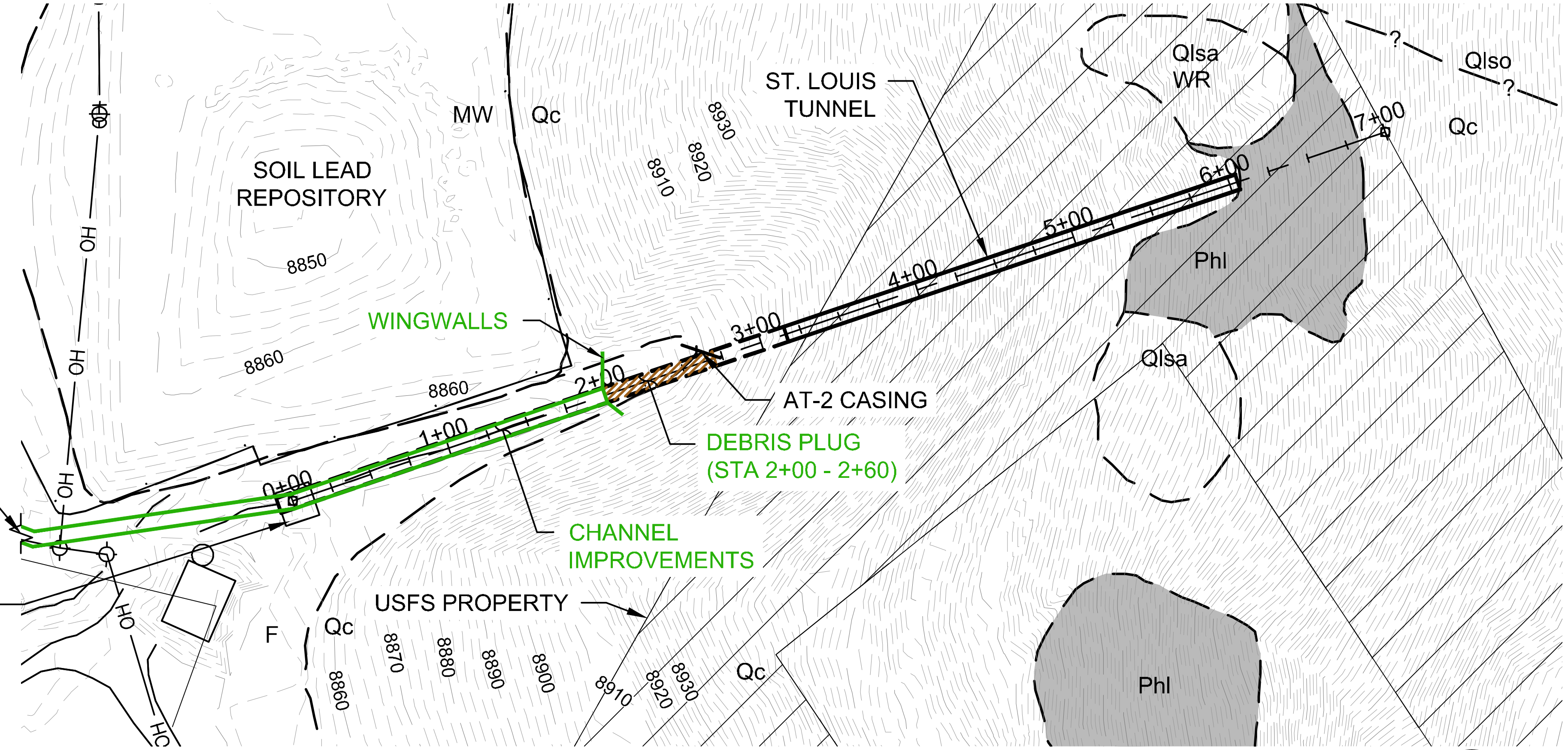
Design Alternative	Max Head (ft)			Max Storage (cf)		
	100-Year	50-Year	25-Year	100-Year	50-Year	25-Year
Alternative 0	>46	>46	>46	>605,000	>605,000	>605,000
Alternative 1	>46	>46	39	>605,000	>605,000	367,874
Alternative 2a	19	17	15	165,112	141,235	121,207
Alternative 2b	13	12	12	102,081	93,519	86,511
Alternative 3 (3 wells)	28	26	24	258,475	236,713	218,418

Exhibits

LEGEND:

F	FILL
MW	MINE WASTE
WR	WASTE ROCK
Phl	HERMOSA FORMATION
Qc	COLLUVIUM
Qlsa	ACTIVE LANDSLIDE DEPOSIT

NOTE:
IF ADDITIONAL RELIEF CAPACITY REQUIRED, INSTALL ADDITIONAL RELIEF WELL(S) PER ALTERNATIVE 2.



PLAN

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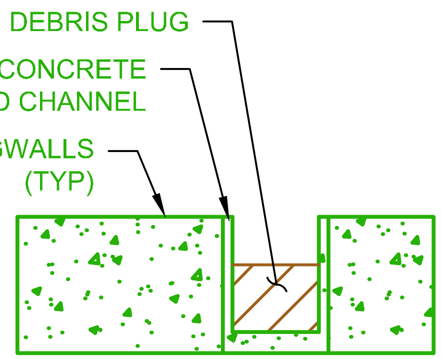
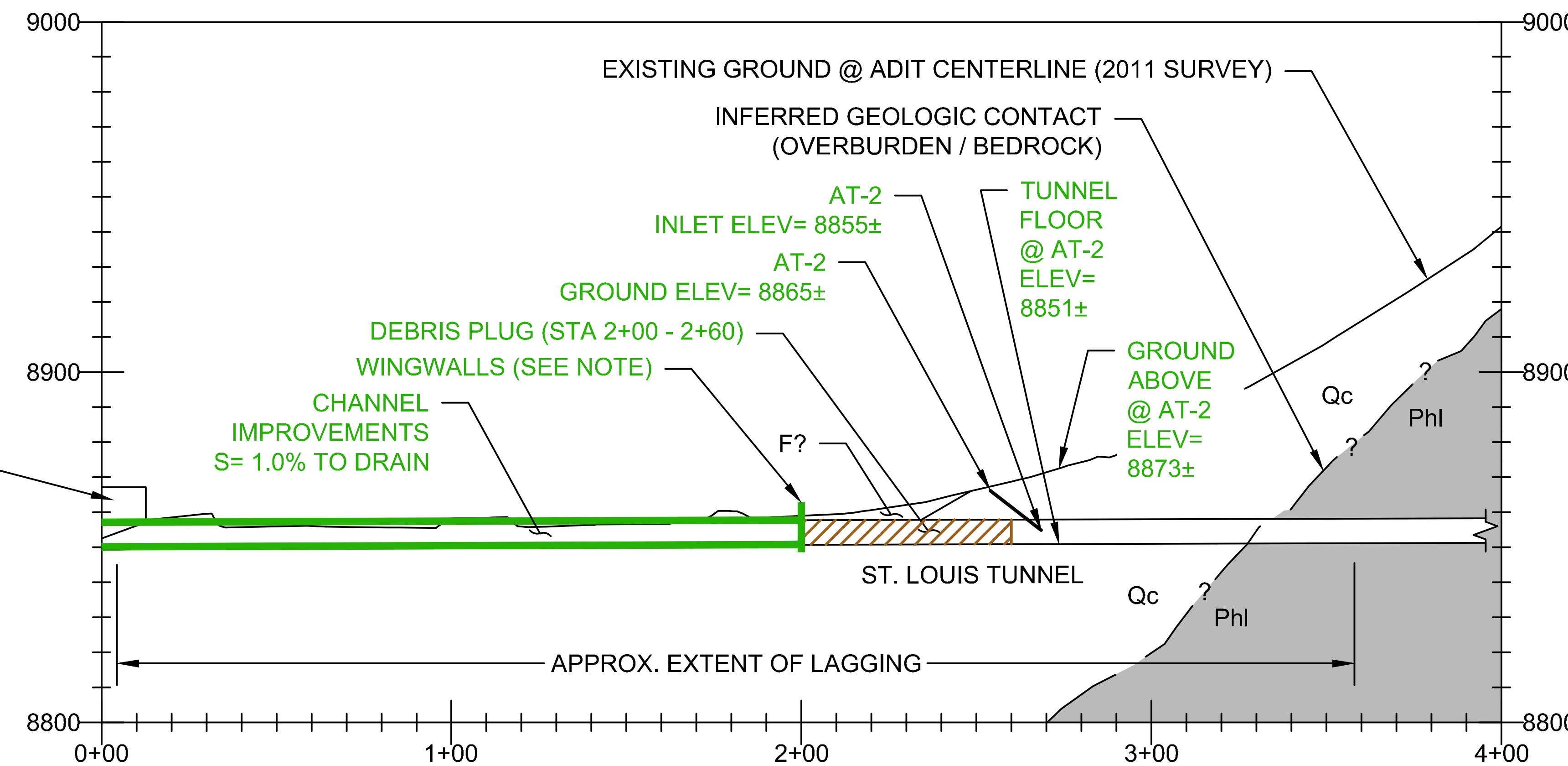


WINGWALLS (LOOKING UPSTREAM)

N.T.S.



PROFILE

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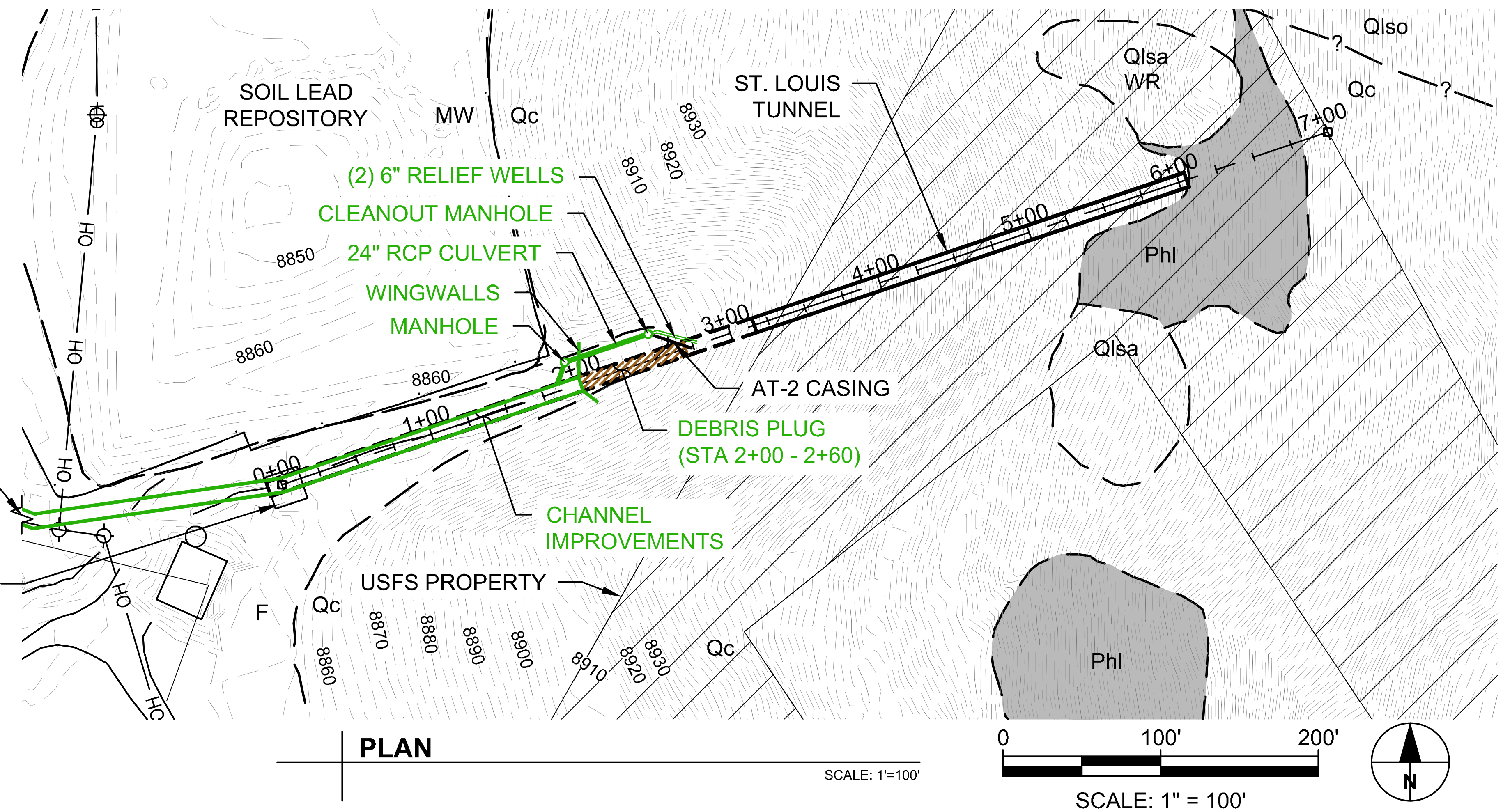
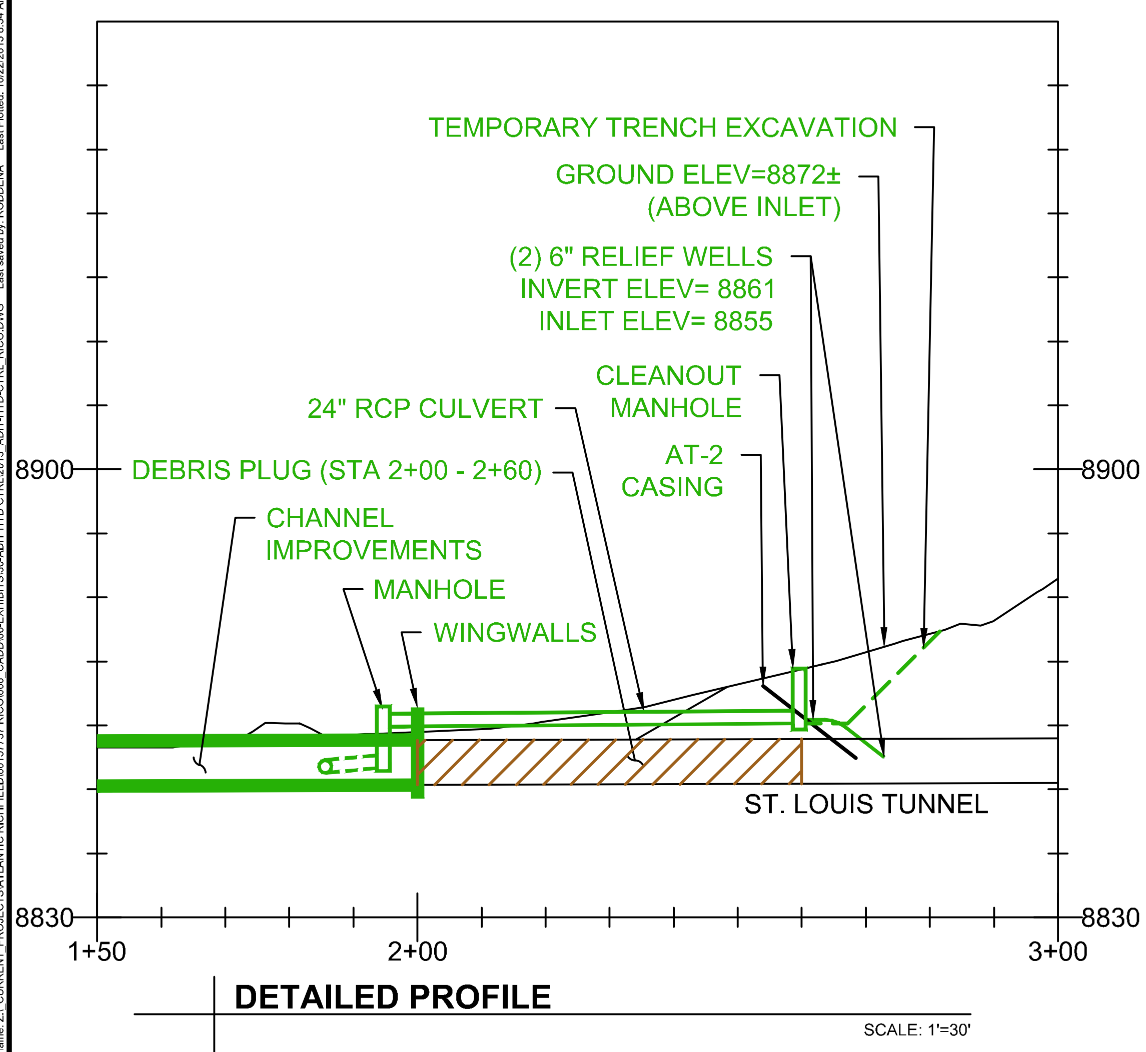
RICO-ARGENTINE SITE-OU01

2013 ADIT HYDRAULIC CONTROL
EXHIBIT 4.1 - ALTERNATIVE #1 (BASE CASE)

LEGEND:

F	FILL
MW	MINE WASTE
WR	WASTE ROCK
Phl	HERMOSA FORMATION
Qc	COLLUVIUM
Qlsa	ACTIVE LANDSLIDE DEPOSIT





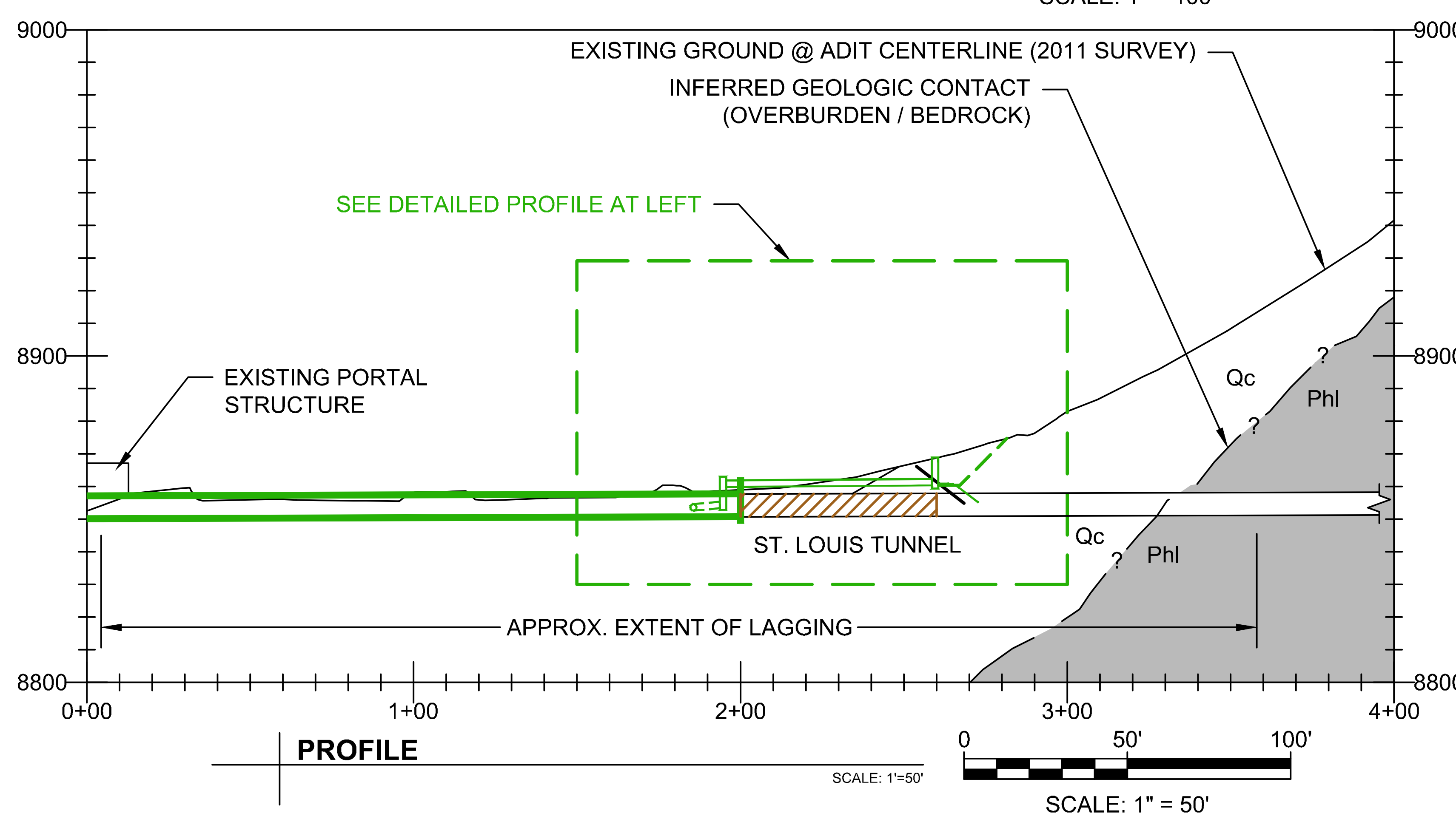


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EXHIBIT 4.2 - ALTERNATIVE #2 (BASE CASE PLUS HORIZONTAL WELL(S) TO RELIEVE TUNNEL HEAD)

LEGEND:

F	FILL
MW	MINE WASTE
WR	WASTE ROCK
Phl	HERMOSA FORMATION
Qc	COLLUVIUM
Qlsa	ACTIVE LANDSLIDE DEPOSIT

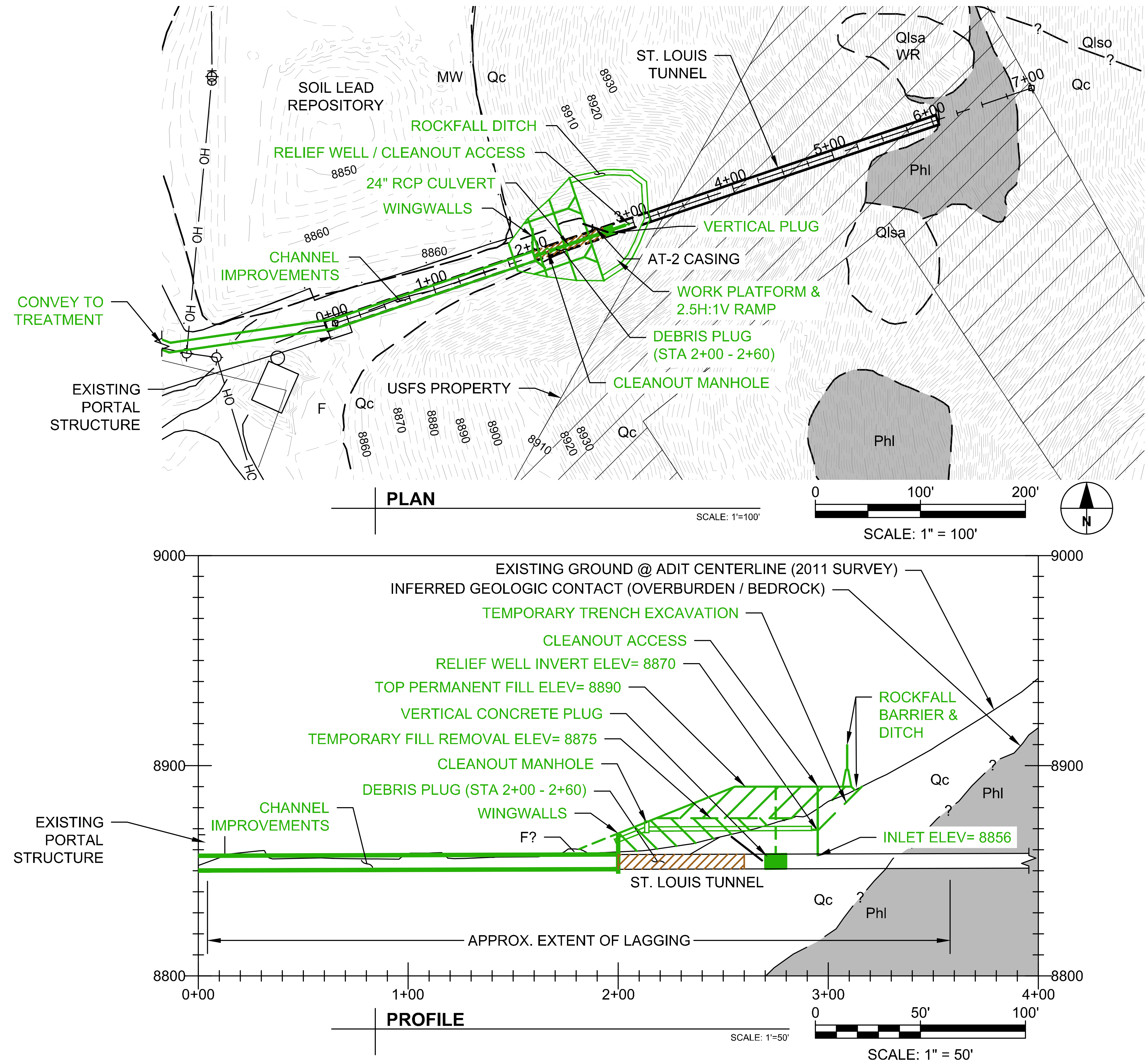

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EXHIBIT 4.3 - ALTERNATIVE #3 (HORIZONTAL WELL DRAIN(S) PLUS PLUGGING OF THE DEBRIS PLUG/TUNNEL)

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EXHIBIT 4.4 - ALTERNATIVE #4 (INTERCEPTION WALL)

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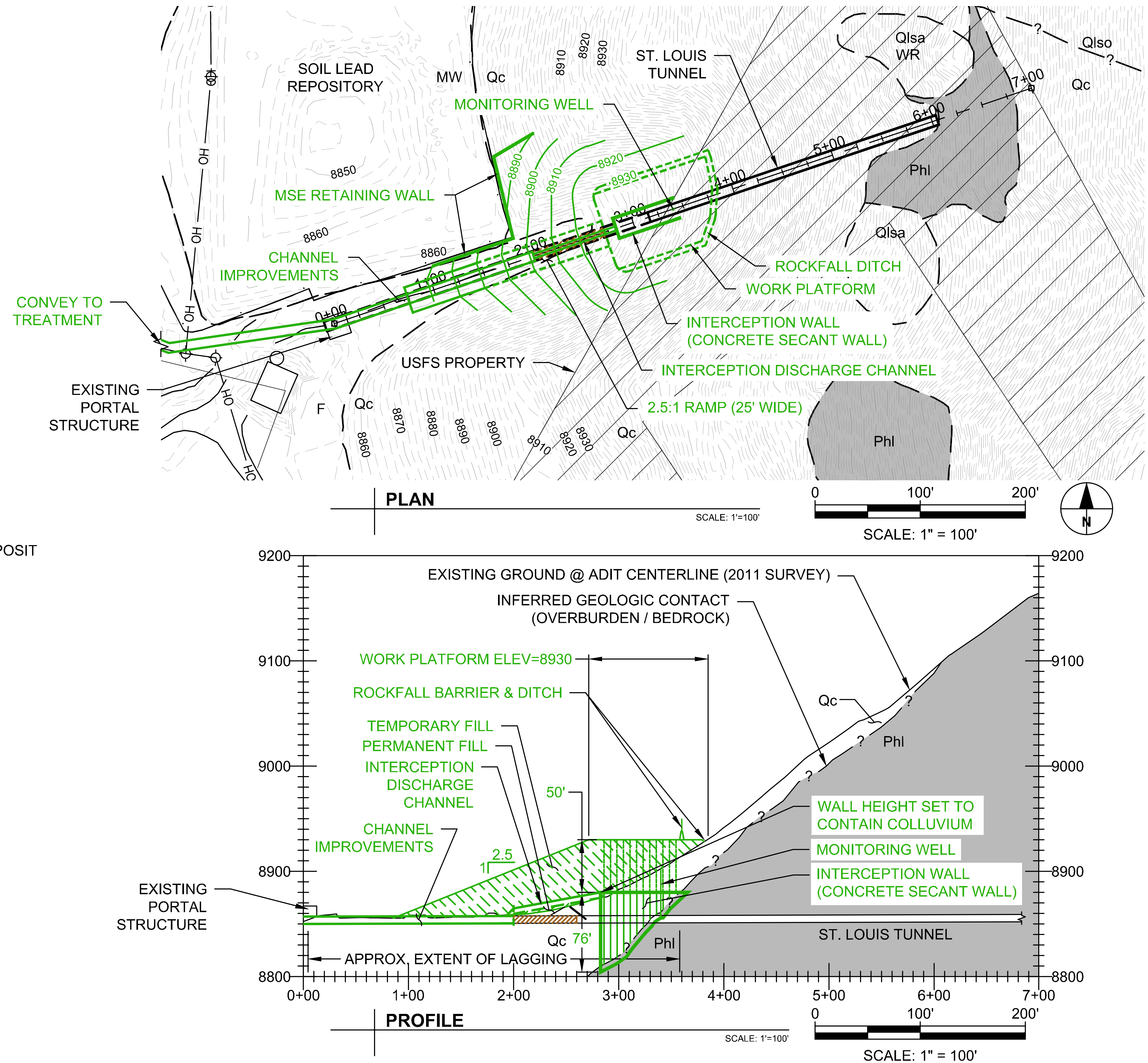
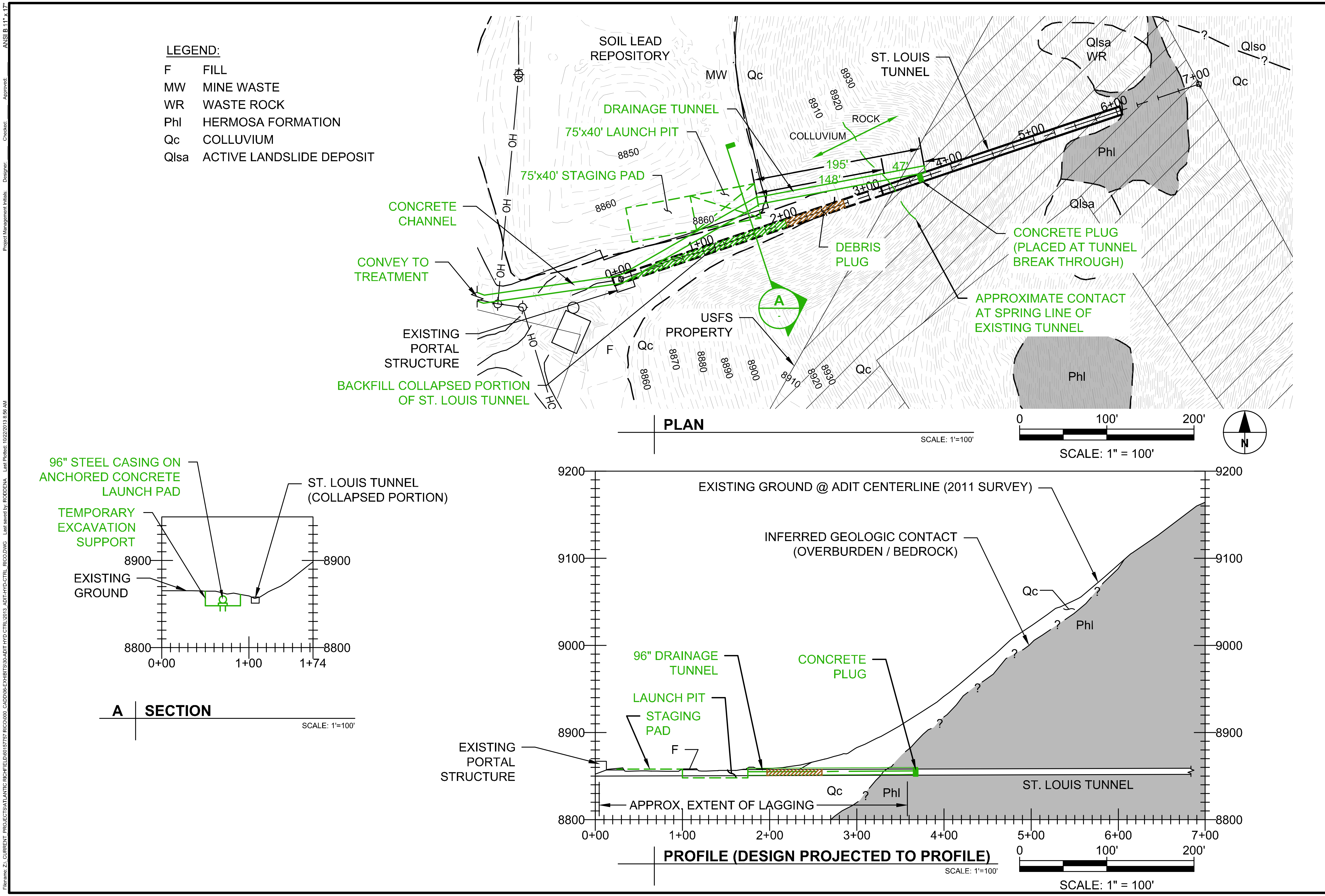


LEGEND:

F	FILL
MW	MINE WASTE
WR	WASTE ROCK
Phl	HERMOSA FORMATION
Qc	COLLUVIUM
Qlsa	ACTIVE LANDSLIDE DEPOSIT



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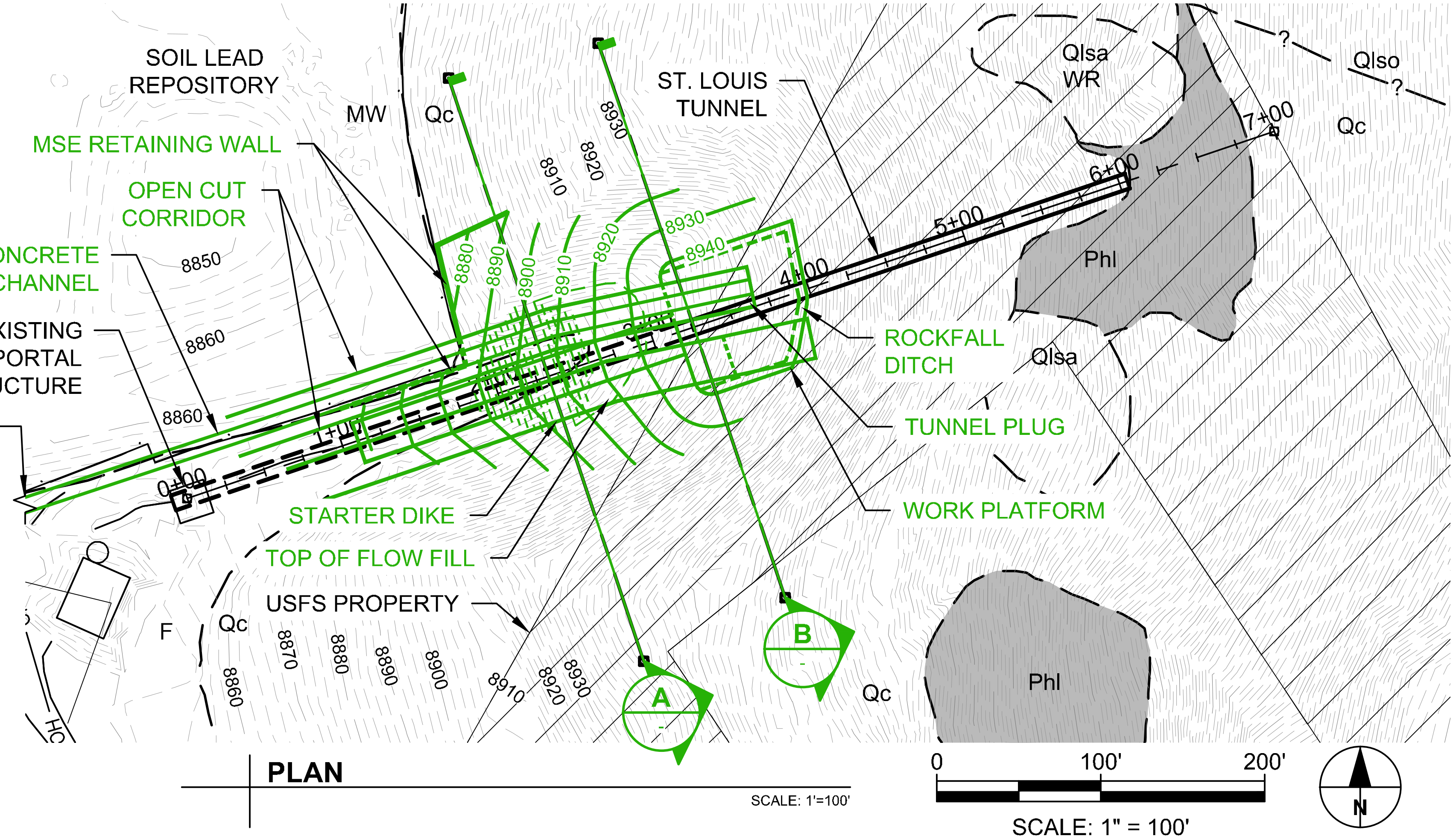
2013 ADIT HYDRAULIC CONTROL

EXHIBIT 4.5 - ALTERNATIVE #5 (TUNNELING)

LEGEND:

F	FILL	Phl	HERMOSA FORMATION
MW	MINE WASTE	Qc	COLLUVIUM
WR	WASTE ROCK	Qlsa	ACTIVE LANDSLIDE DEPOSIT



PLAN

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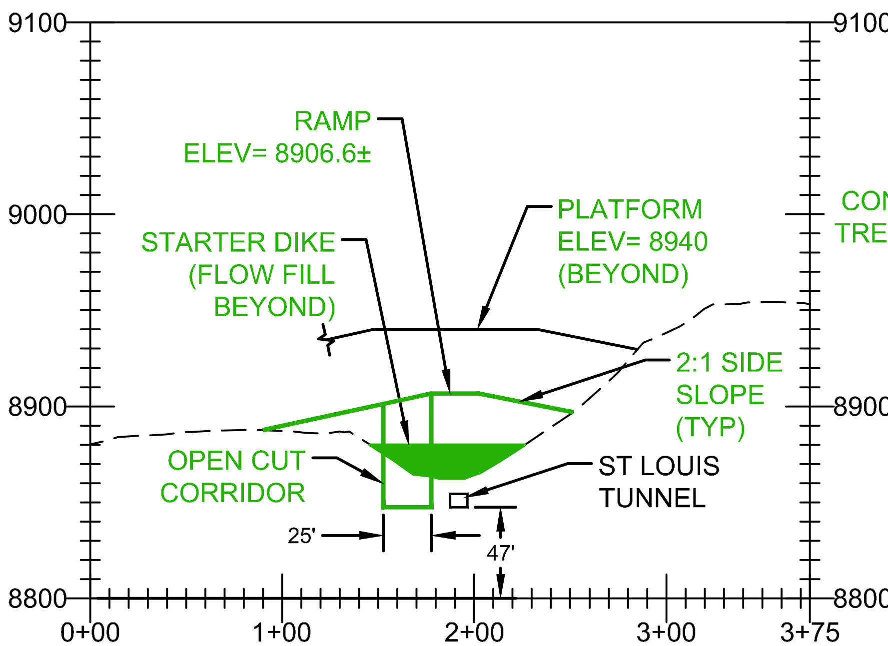
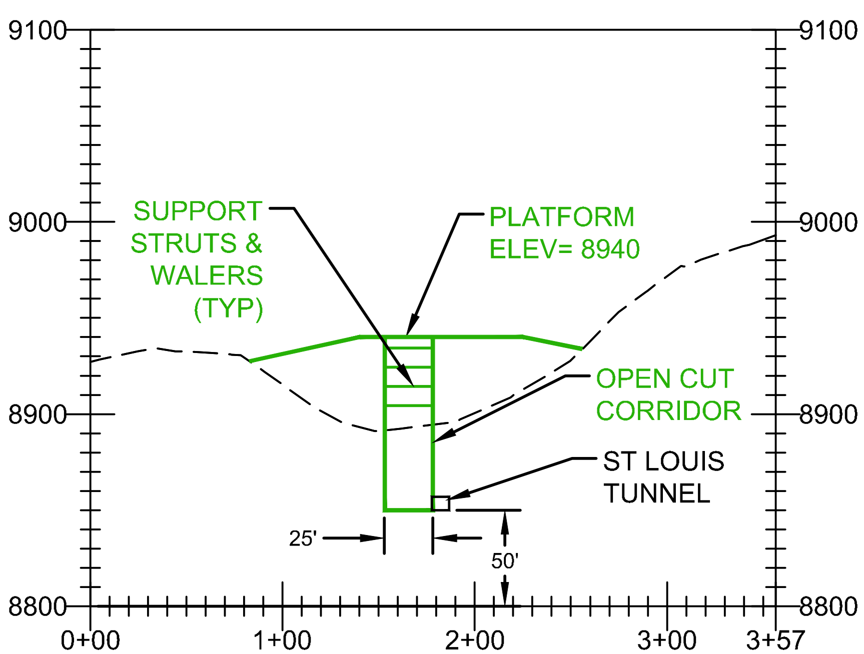


A SECTION

SCALE: 1"=100'



B SECTION

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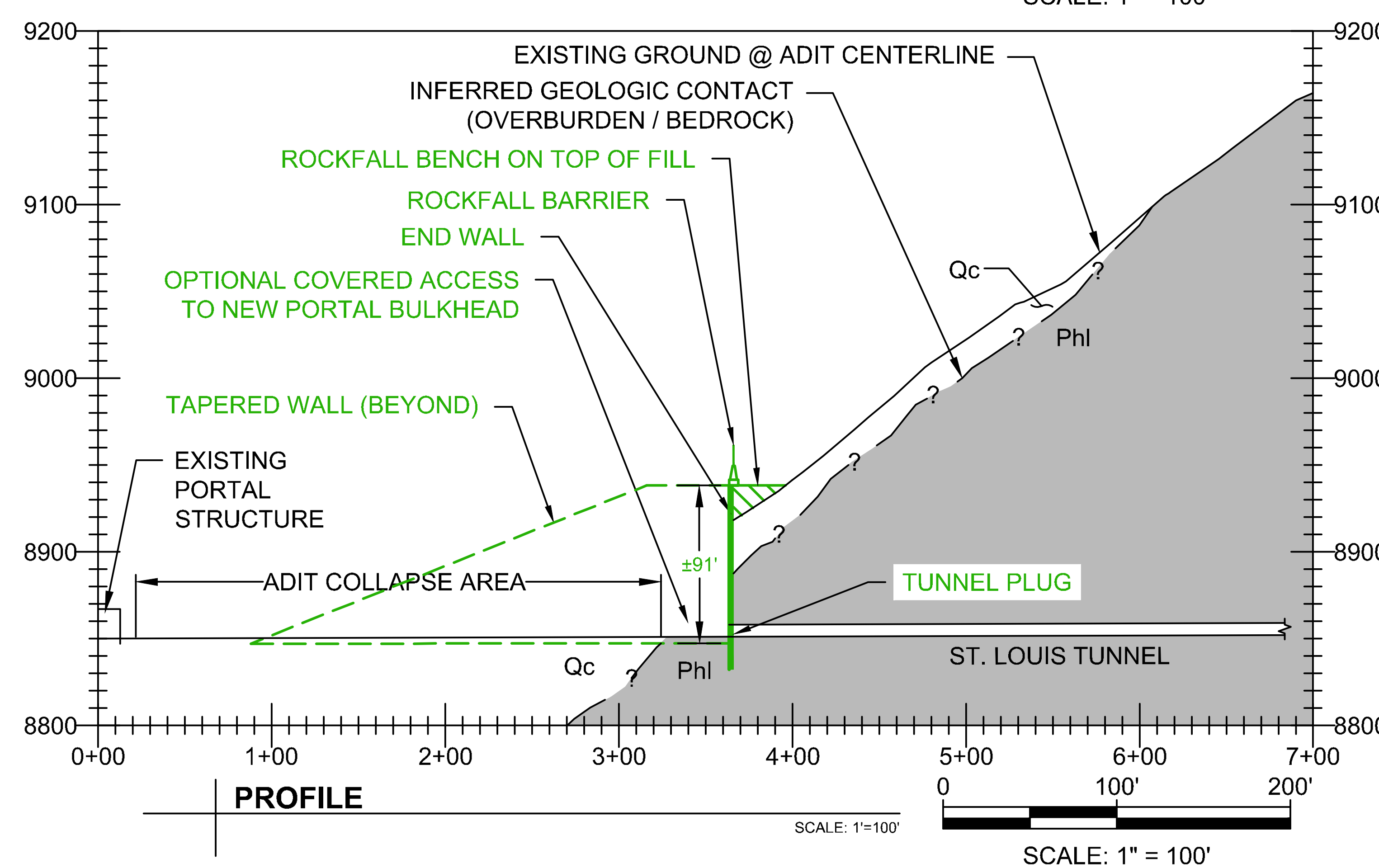

PROFILE

SCALE: 1"=100'

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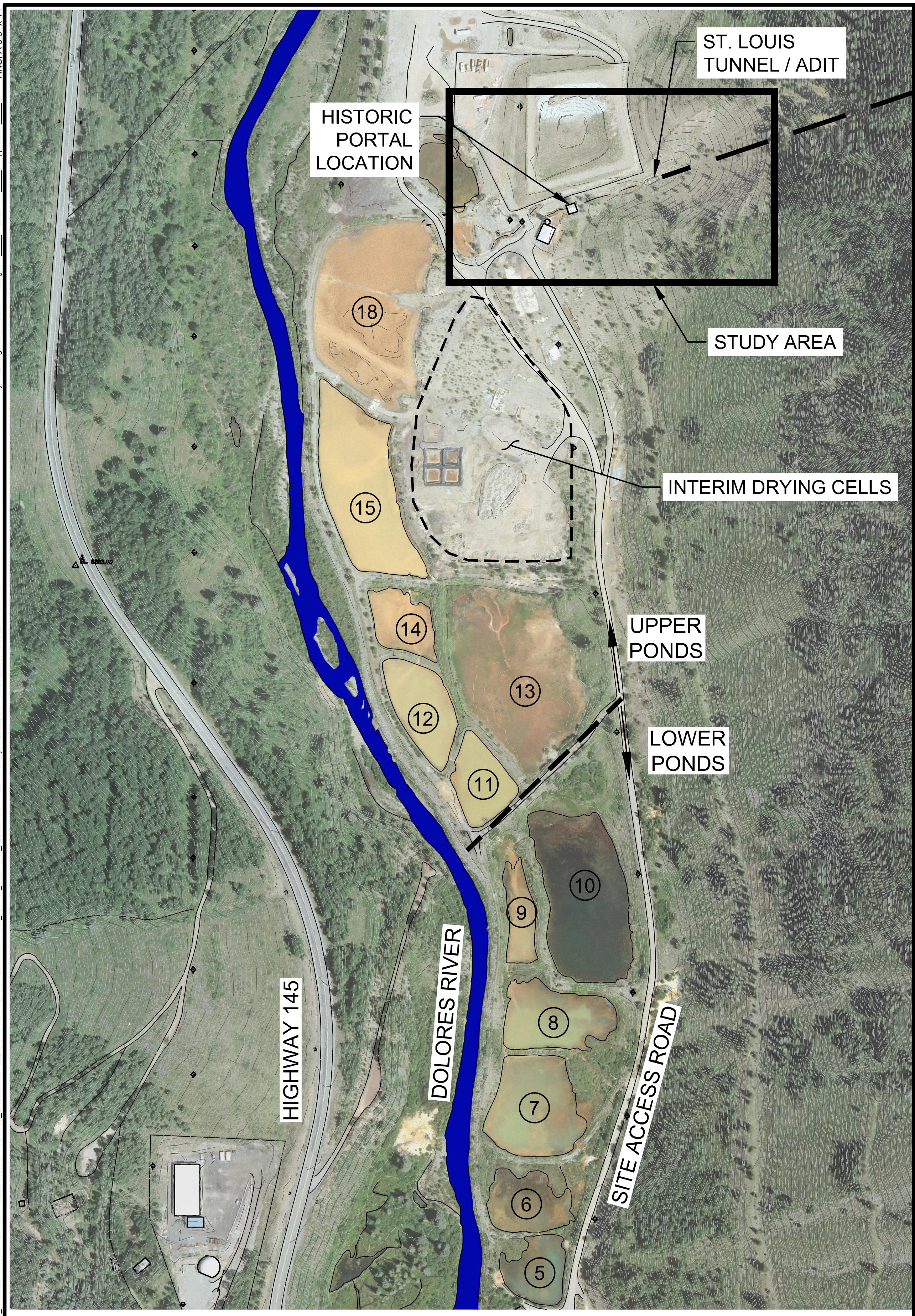
2013 ADIT HYDRAULIC CONTROL

EXHIBIT 4.6 - ALTERNATIVE #6 (RETAINING WALLS)

Figures



SITE MAP

Scale: 1'=200'

0 200'

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RICO-ARGENTINE SITE-OU01

ADIT HYDRAULIC CONTROL

FIGURE 1.1 - SITE MAP

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Filename: Z:_CURRENT_PROJECTS\ATLANTIC RICHFIELD\60157757 RICO\000_CADD\06-EXHIBITS\39-2013-ADIT-PDR\2013-ADIT-PDR_FIG-1.1_SITE-MAP_RICO.DWG Last saved by: RODDENA Last Plotted: 10/28/2013 10:47 AM Project Management Initials: Designer: Checked: Approved: ANSI A 8.5" x 11"

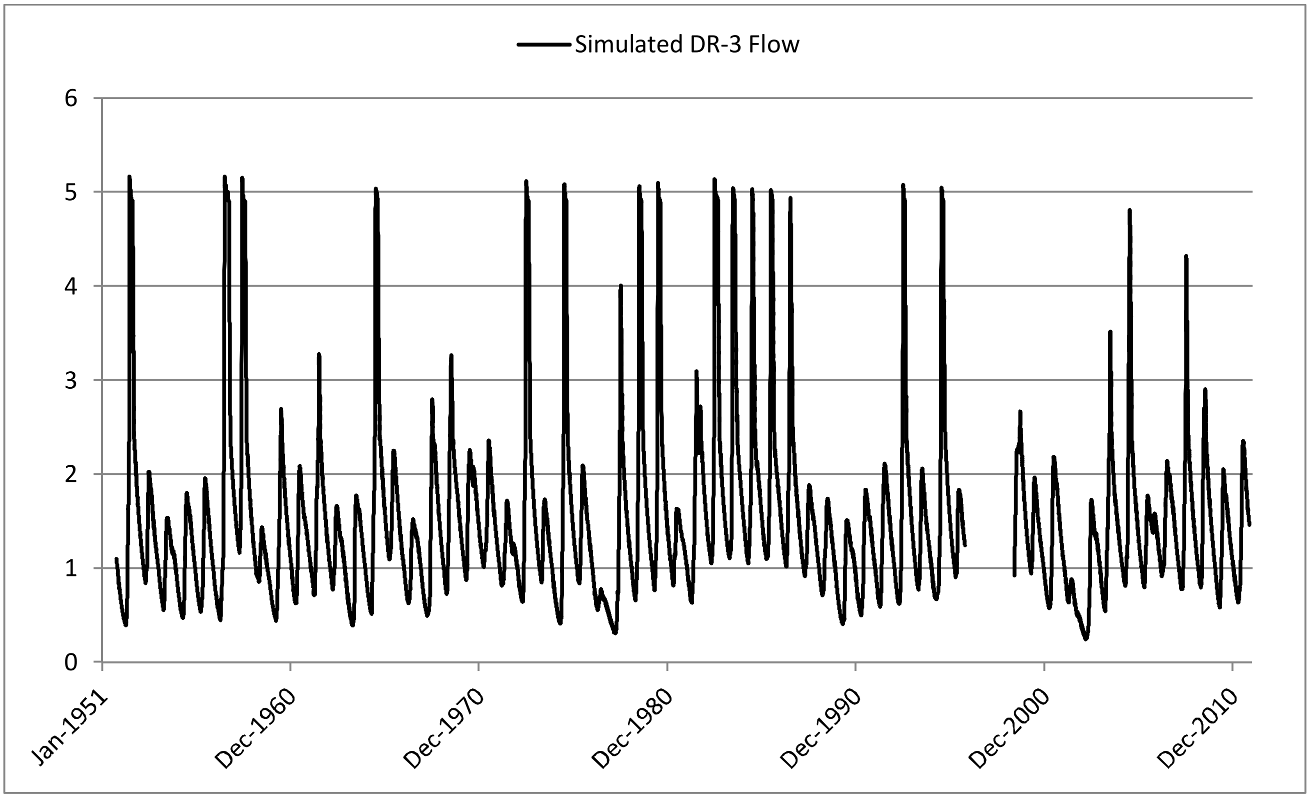


Figure 5.1: Predicted DR-3 flow rate based on the Rico Underground Workings Hydraulic Model.

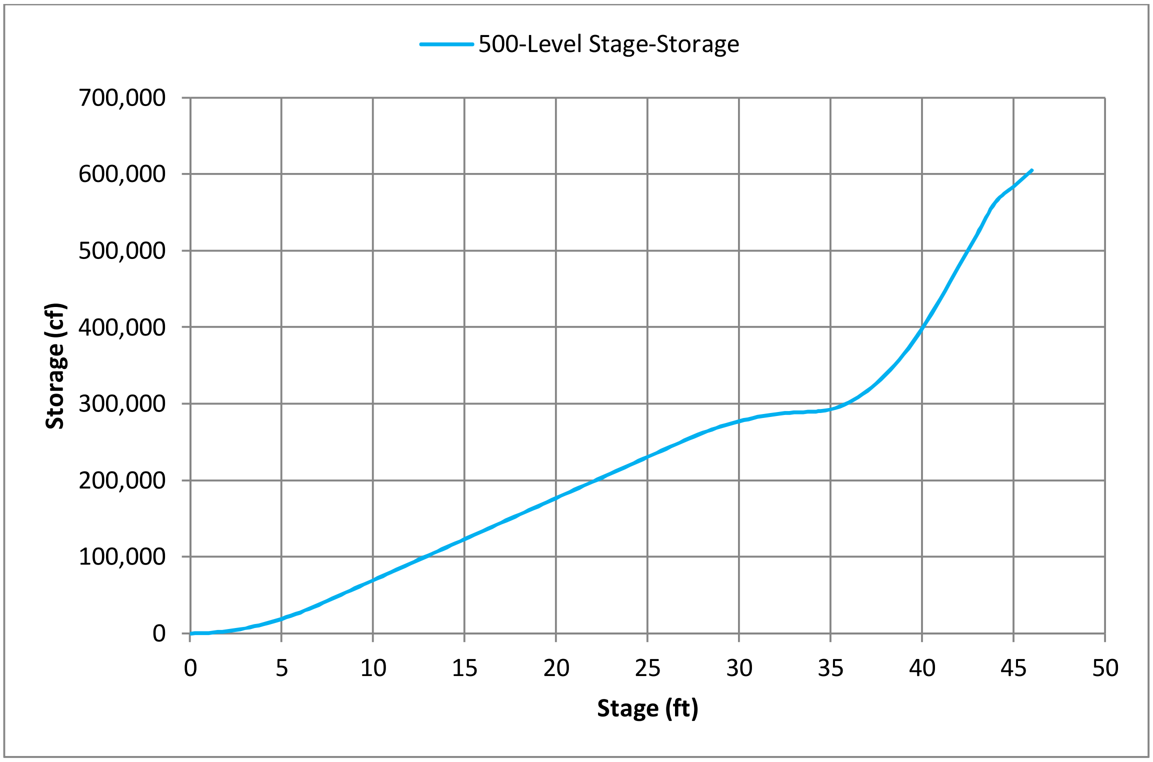


Figure 5.2: Stage-storage relationship for 500 Level underground workings.

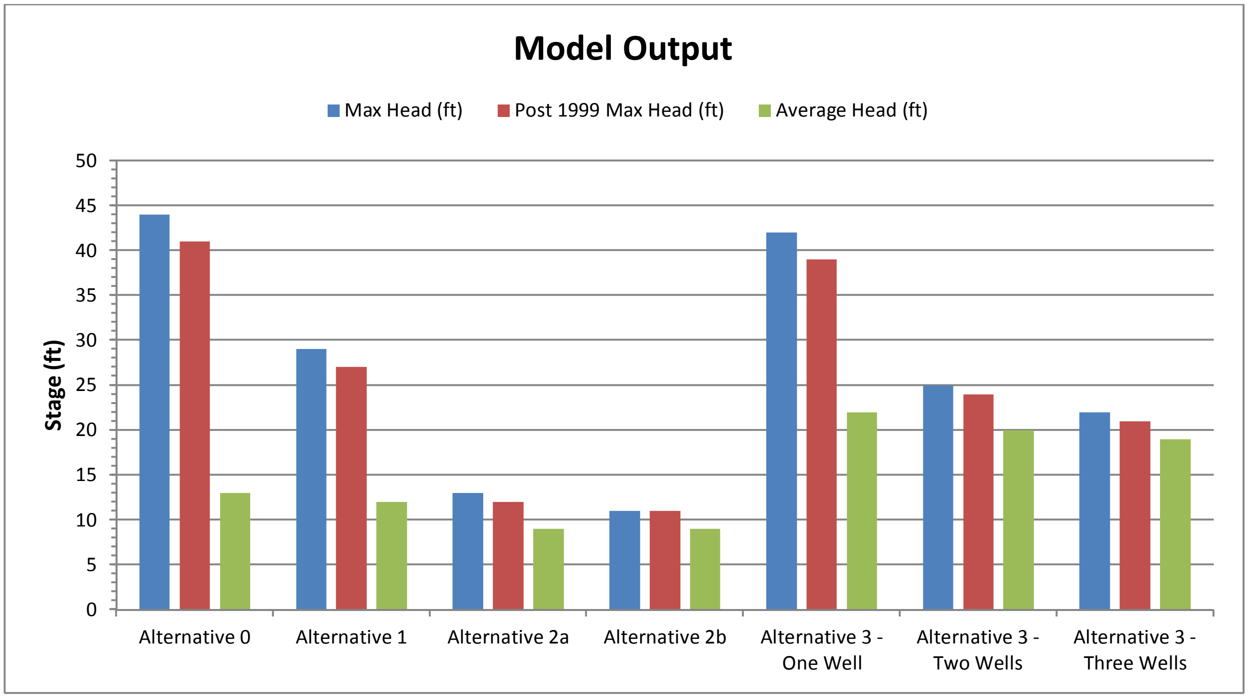


Figure 5.3: Hydraulic model results for Alternatives 0 through 3, showing max and average head upstream of the debris plug.

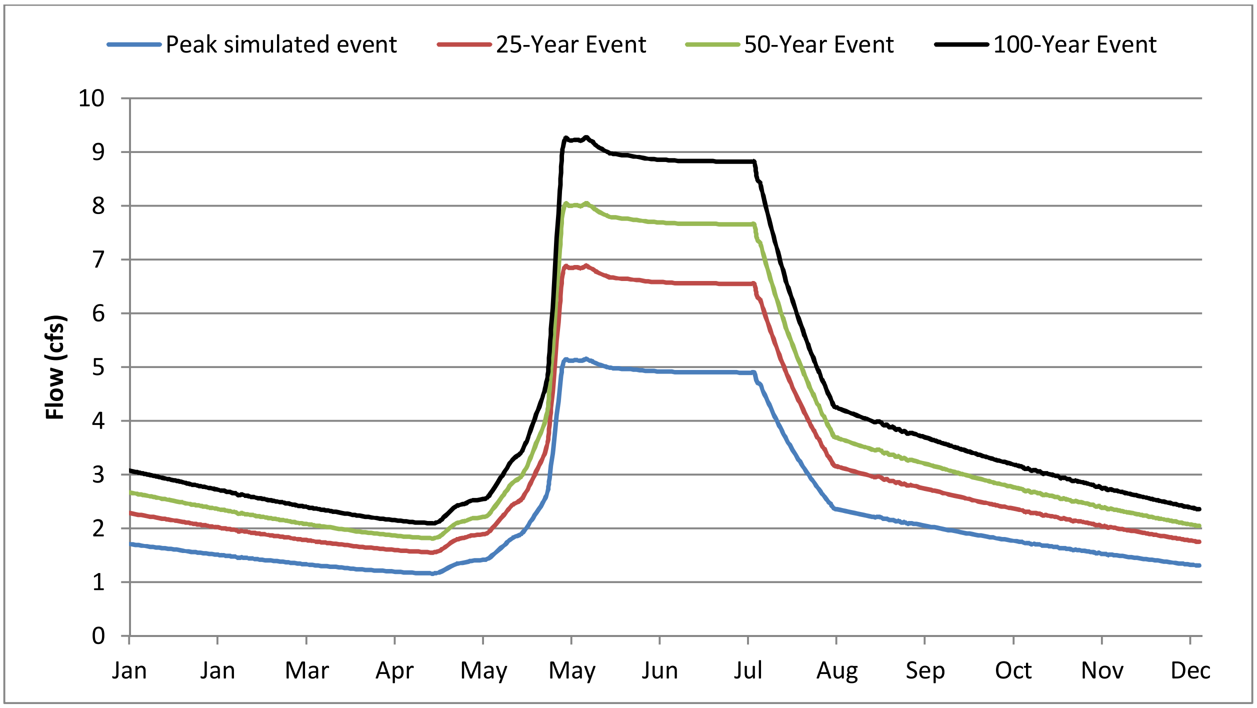


Figure 5.4: Constructed recurrence interval event hydrographs and peak year simulated hydrograph.

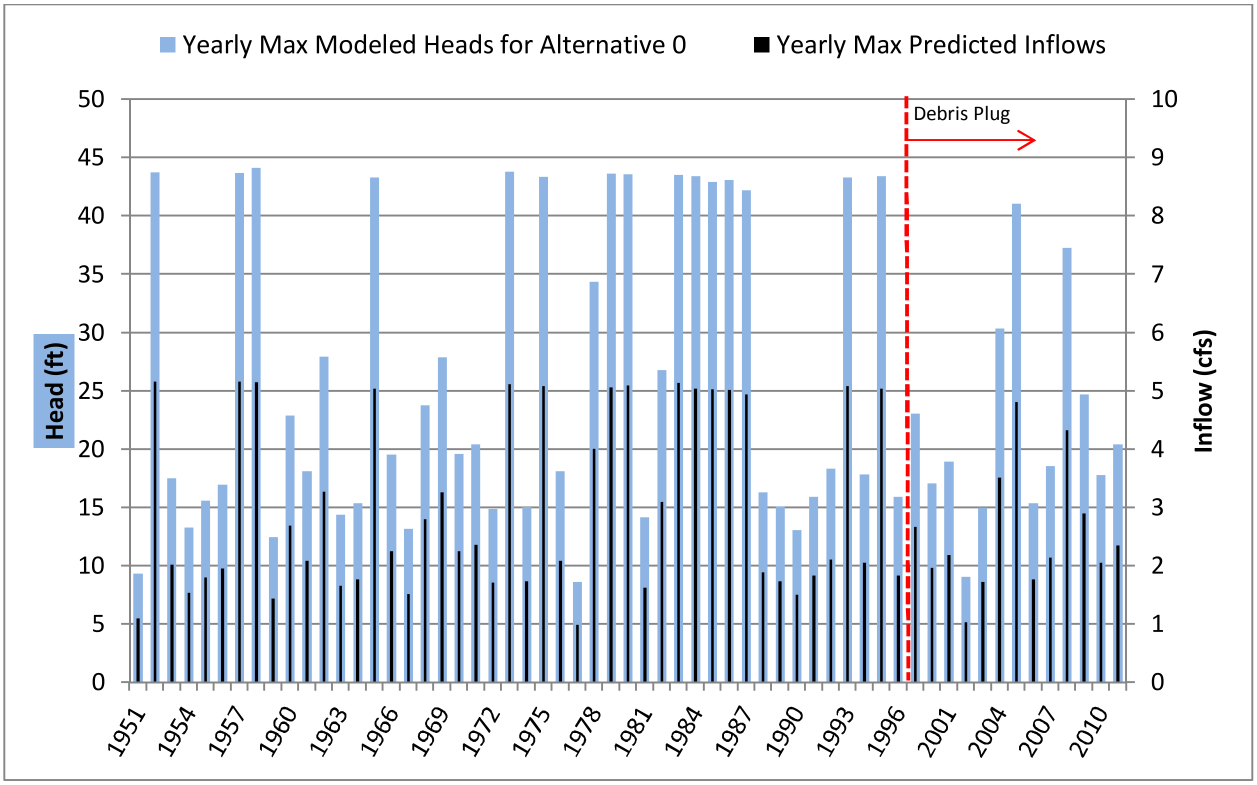


Figure 5.5: Maximum yearly inflows based on the Rico Mine Site Hydraulic Model and the equivalent maximum yearly heads based on the hydraulic routing model for Alternative 0.

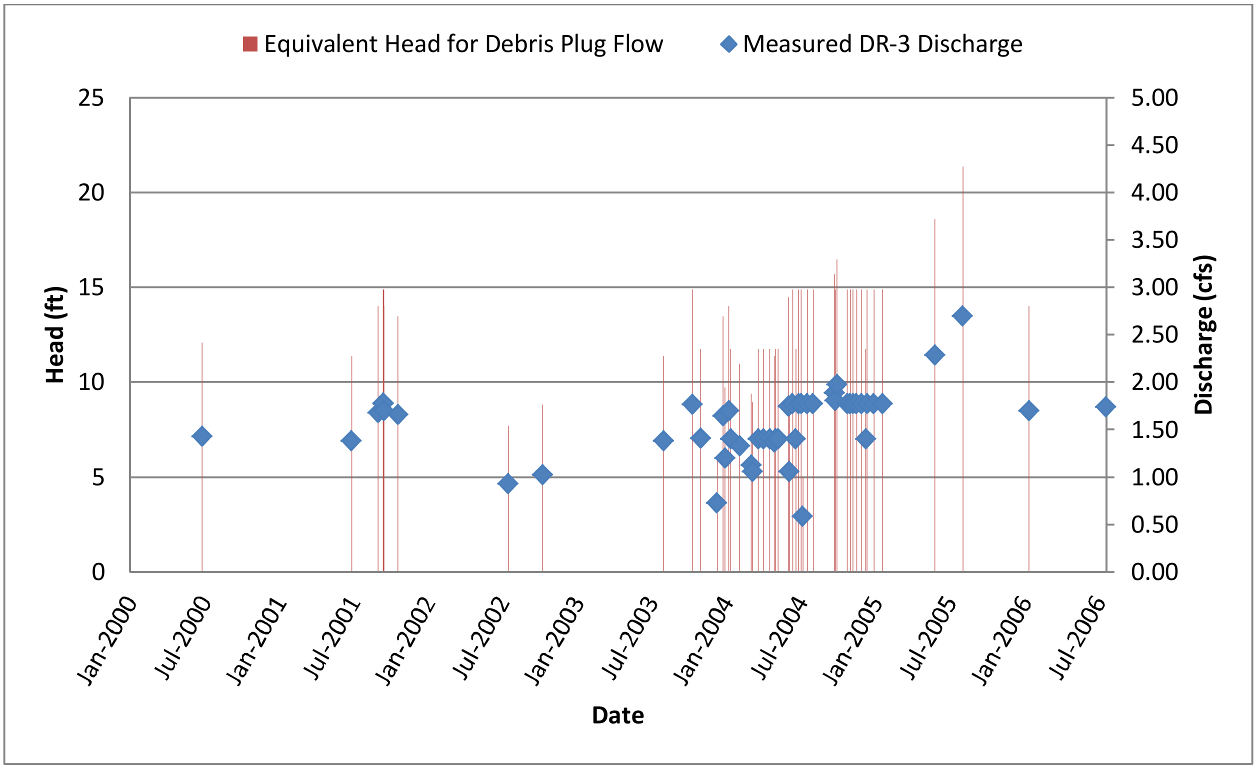


Figure 5.6: Heads experienced by current debris plug based on measured flow rates at DR-3 and equivalent heads from Alternative 0 hydraulic model results.

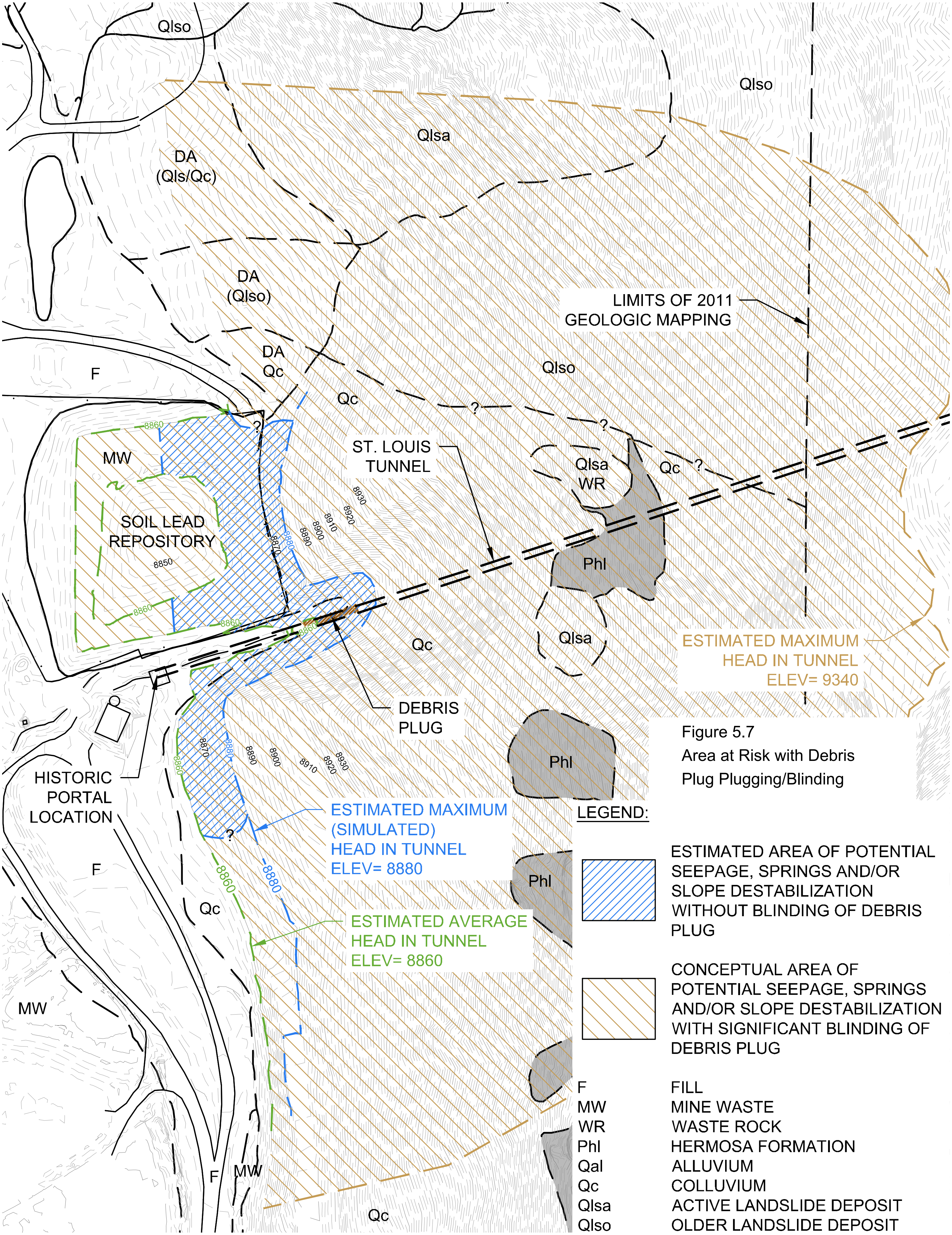


Figure 5.7
Area at Risk with Debris
Plug Plugging/Blinding

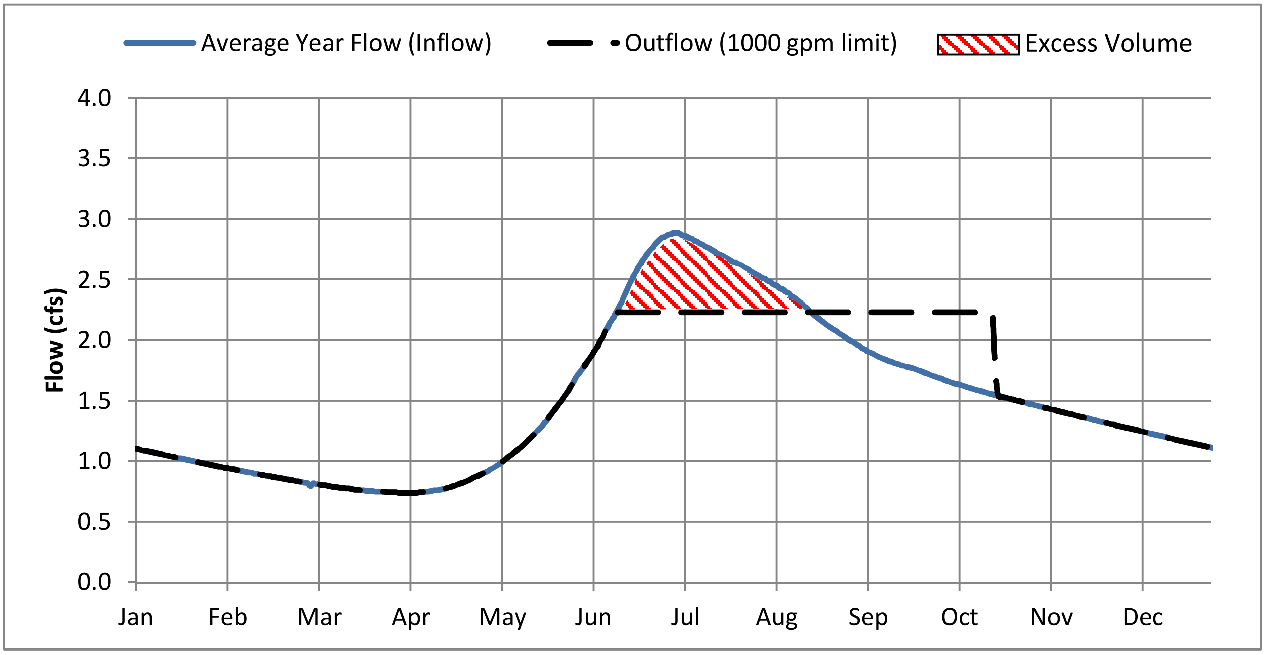


Figure 5.8: Max storage attenuation analysis inflow-outflow hydrograph

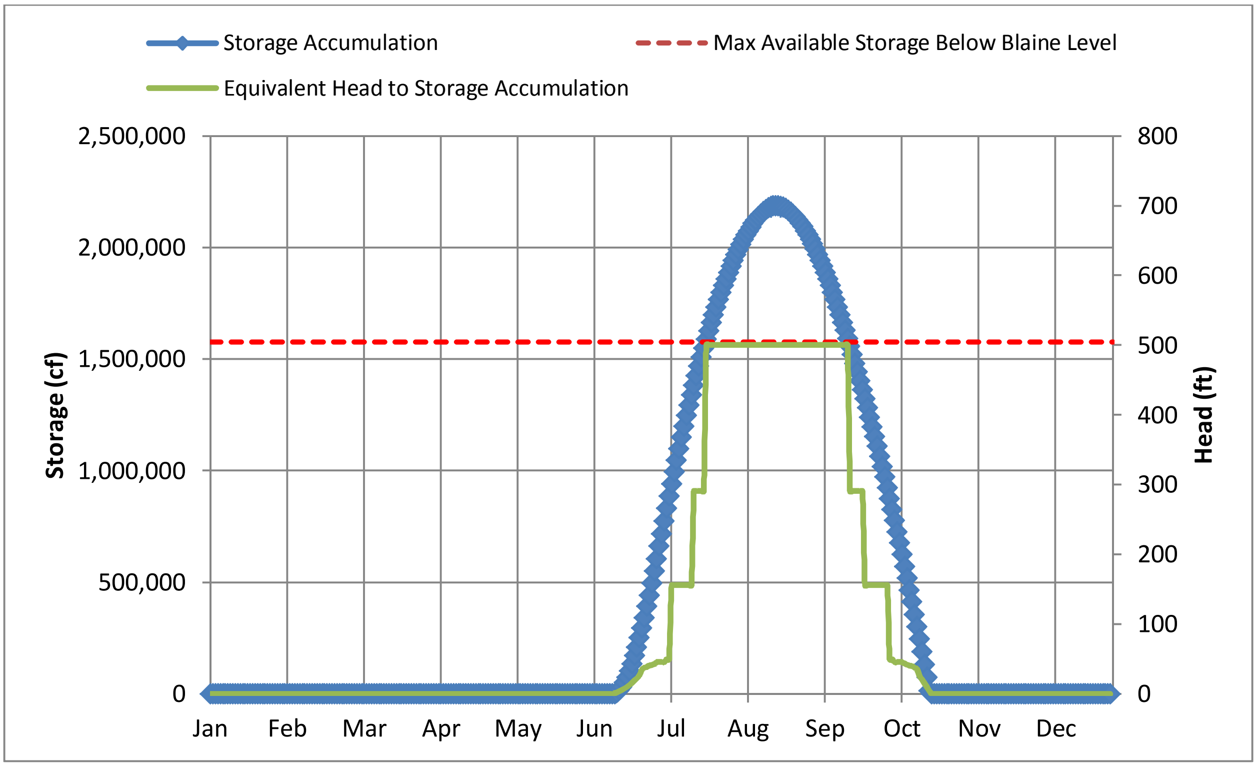


Figure 5.9: Max storage attenuation analysis storage and head accumulation results

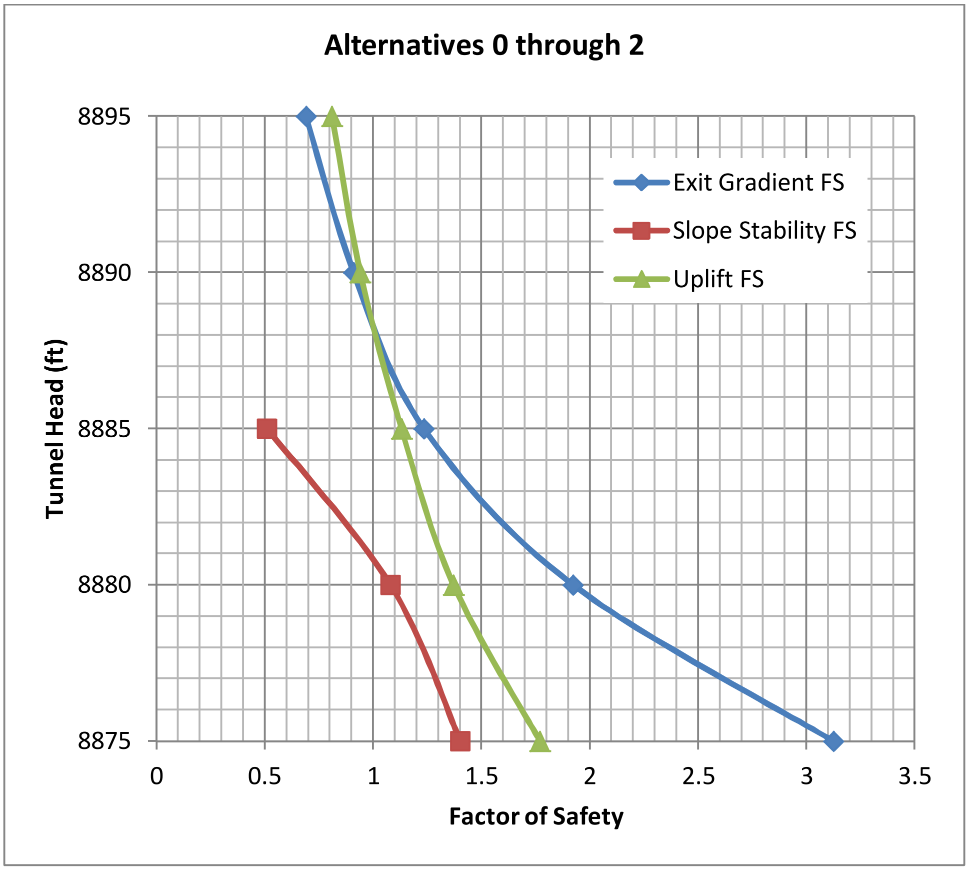


Figure 6.1: Factor of safety results for Alternatives 0-2.

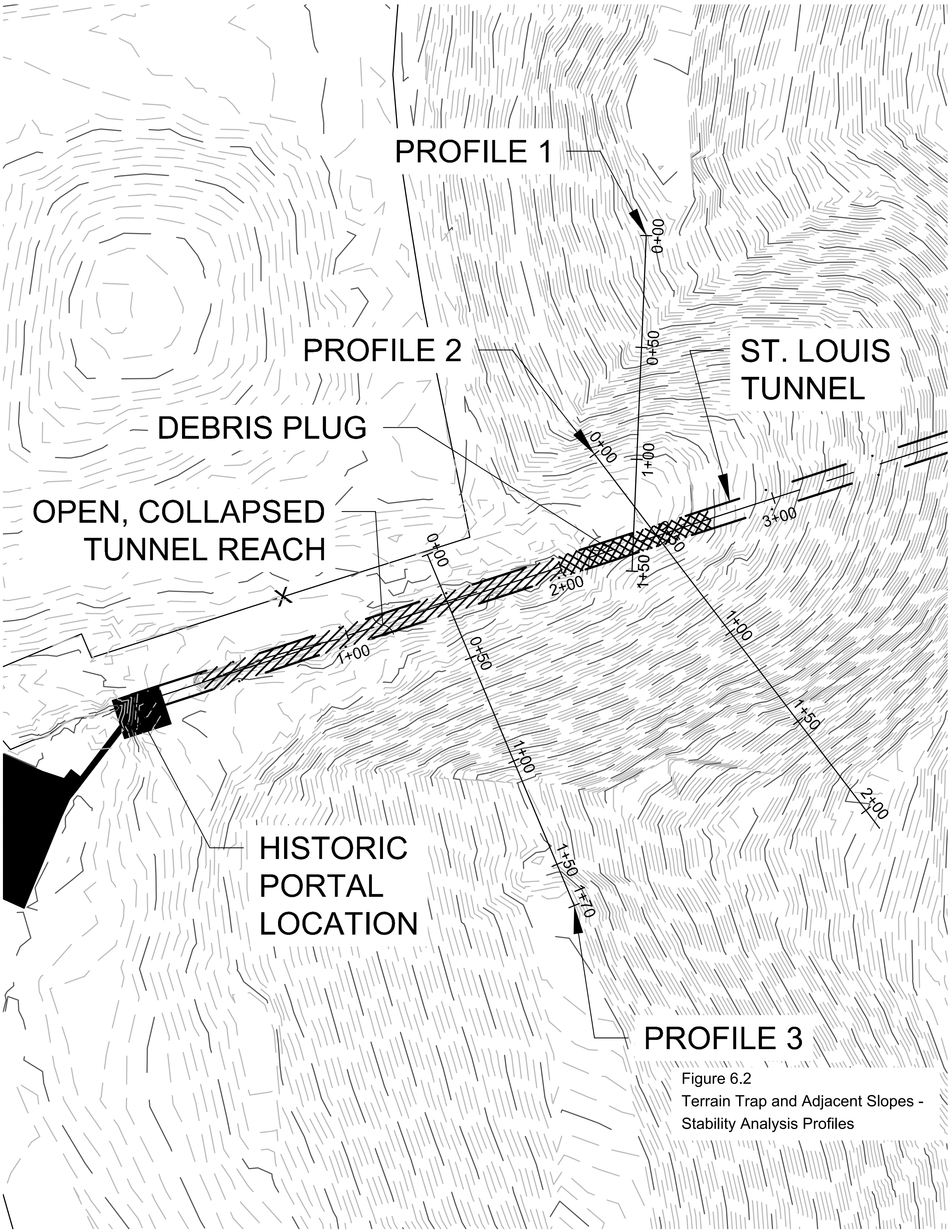


Figure 6.2
Terrain Trap and Adjacent Slopes - Stability Analysis Profiles

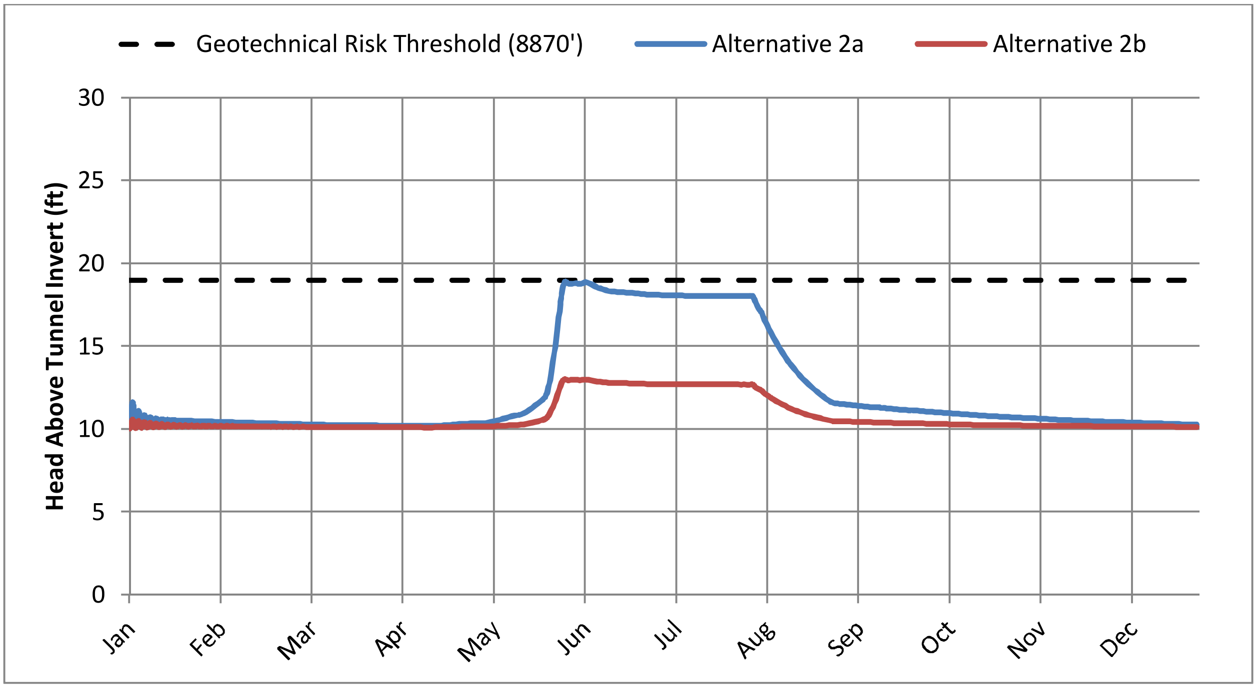


Figure 7.1: Head above tunnel invert for the 100-year event under Alternatives 2a and 2b.

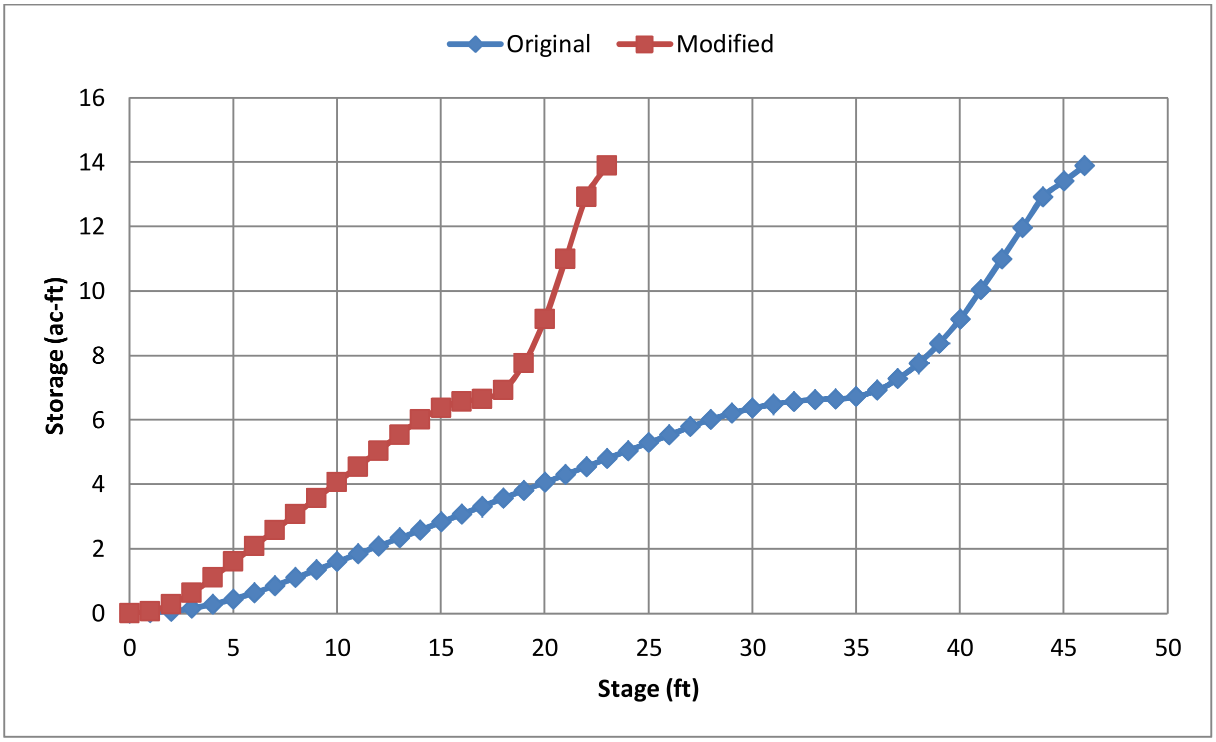
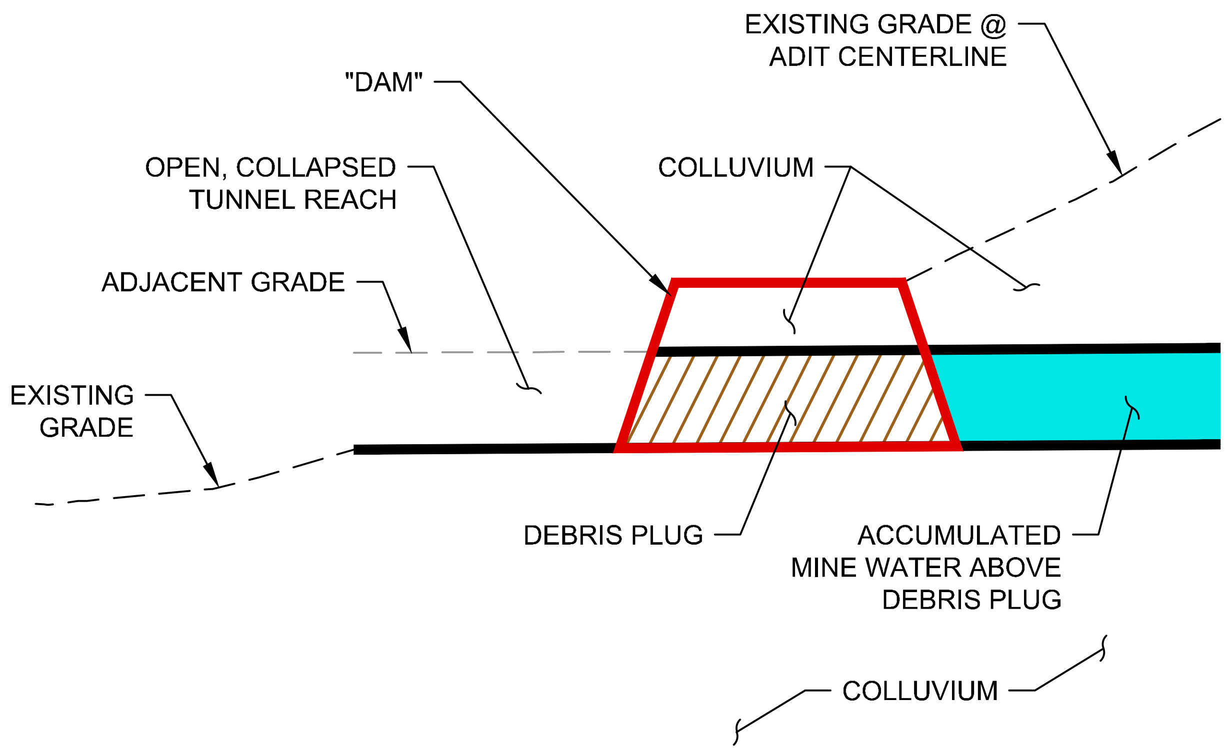


Figure 7.2: Modified dam breach stage capacity curve.



DAM BREACH MODEL PROFILE VIEW

Scale: N.T.S.

Figure 7.3
Dam Breach Model Profile

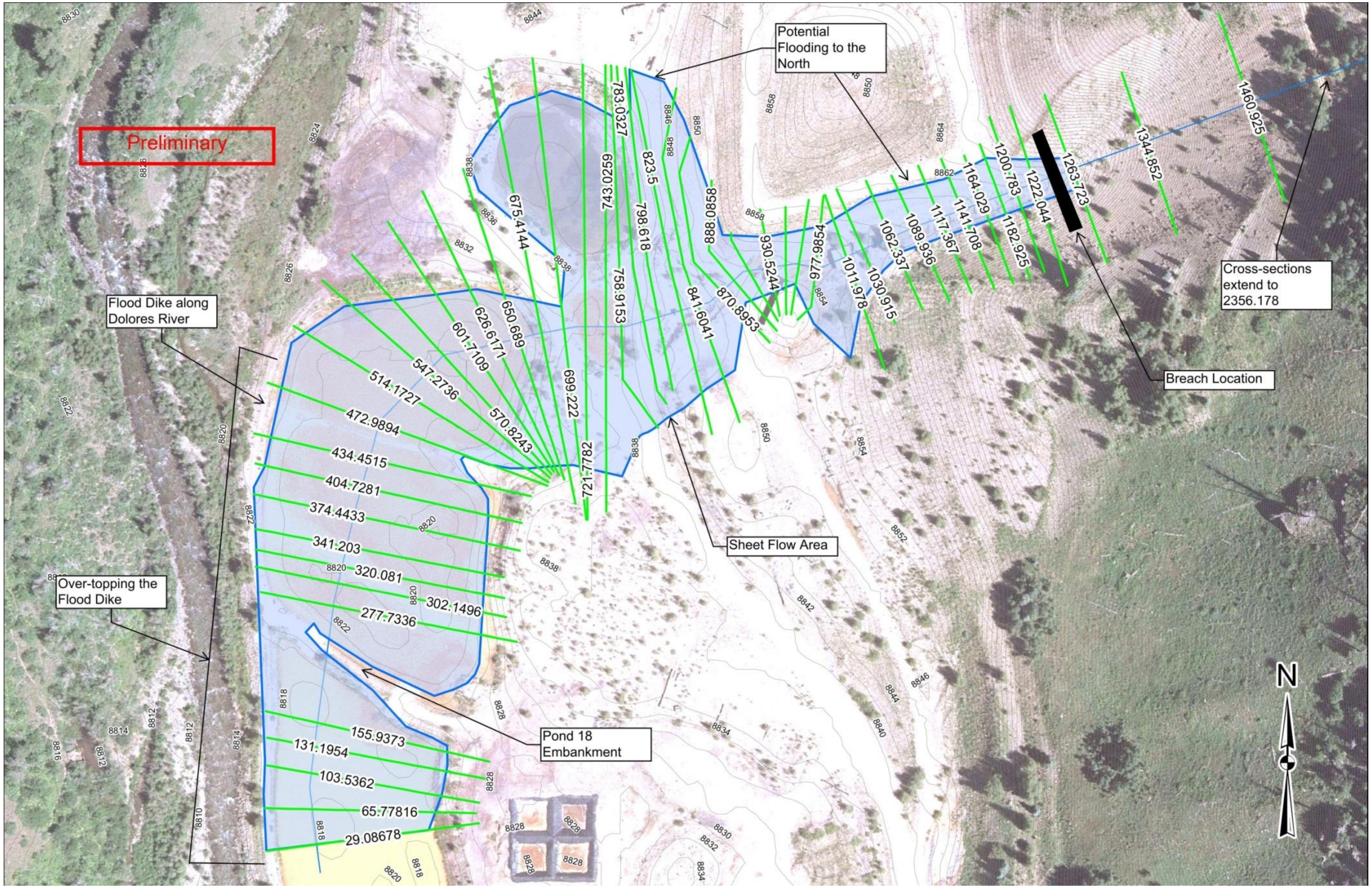


Figure 7.4A: Scenario 1 debris plug failure flood inundation limits for Rico St. Louis Ponds area.

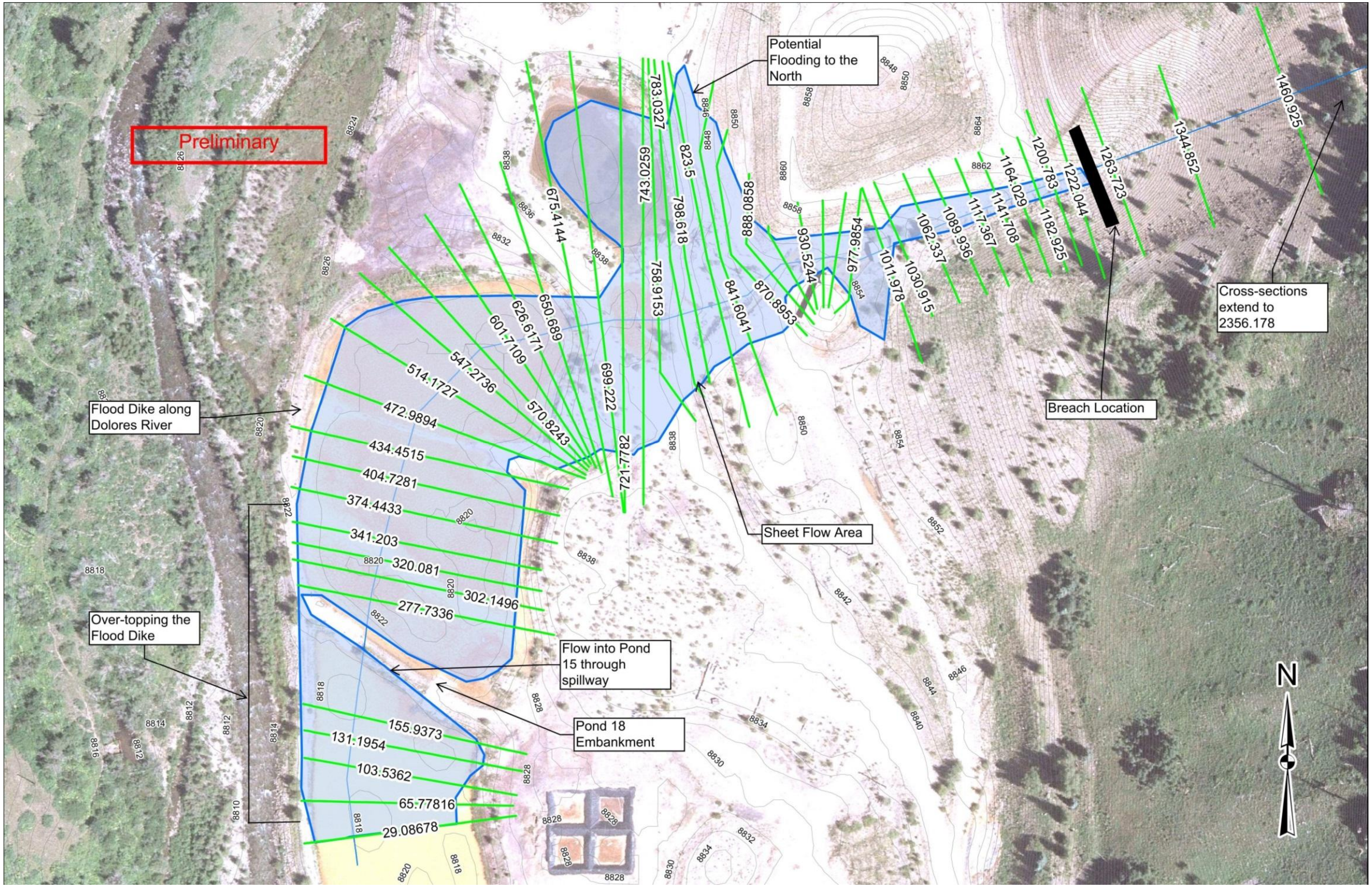


Figure 7.4B: Scenario 2 debris plug failure flood inundation limits for Rico St. Louis Ponds area.

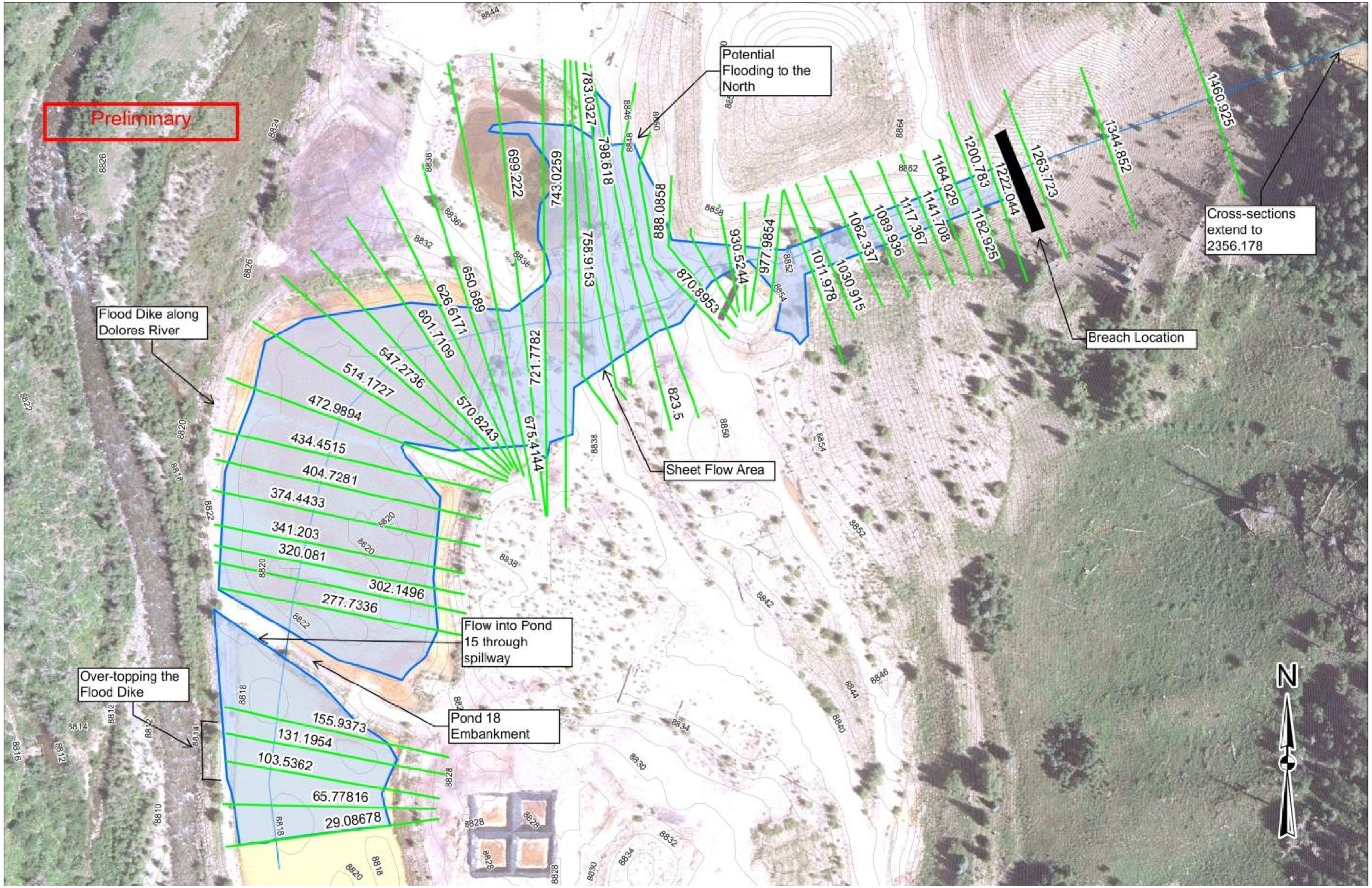


Figure 7.4C: Scenario 3 debris plug failure flood inundation limits for Rico St. Louis Ponds area.

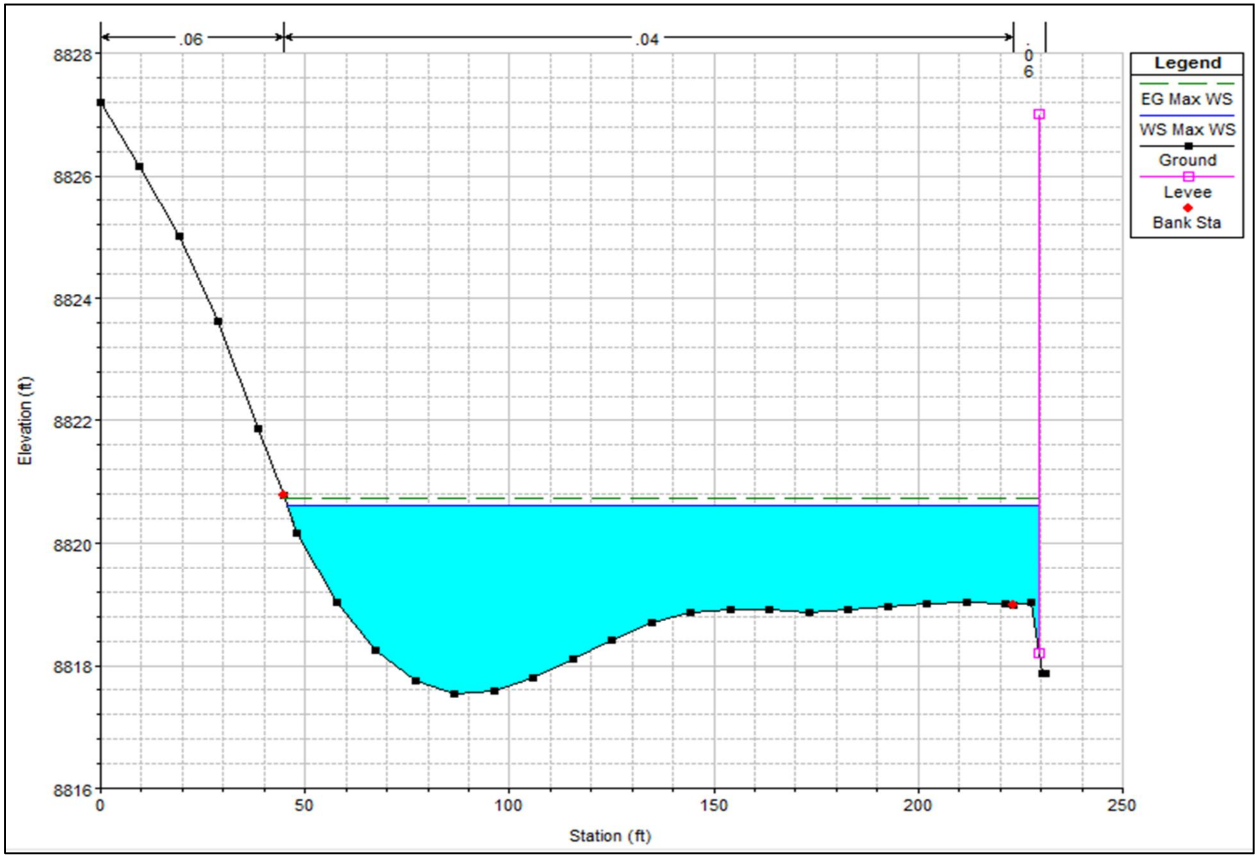


Figure 7.5A: Scenario 1 - Pond 15 Cross-Section 131.1

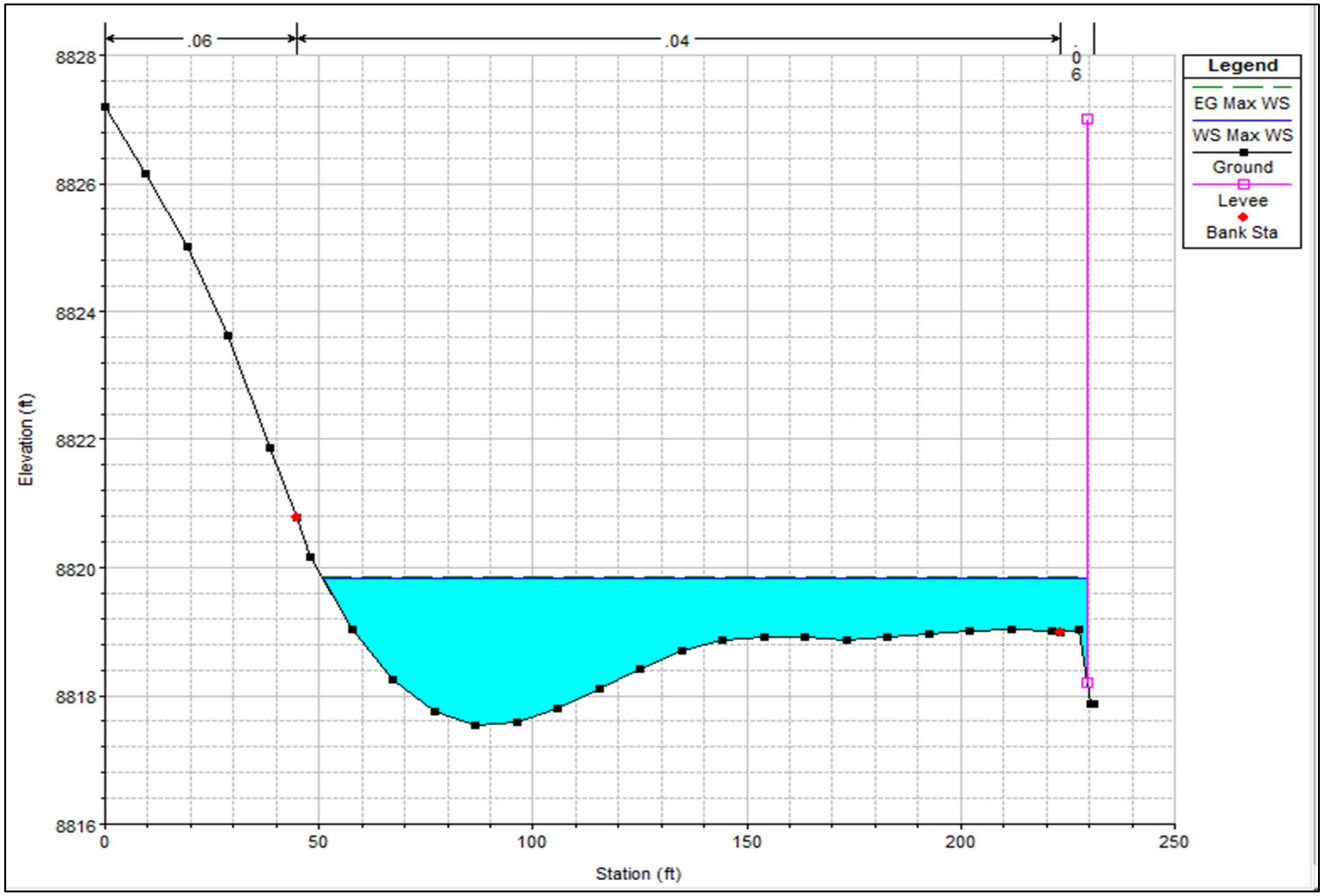


Figure 7.5B: Scenario 2 - Pond 15 Cross-Section 131.1

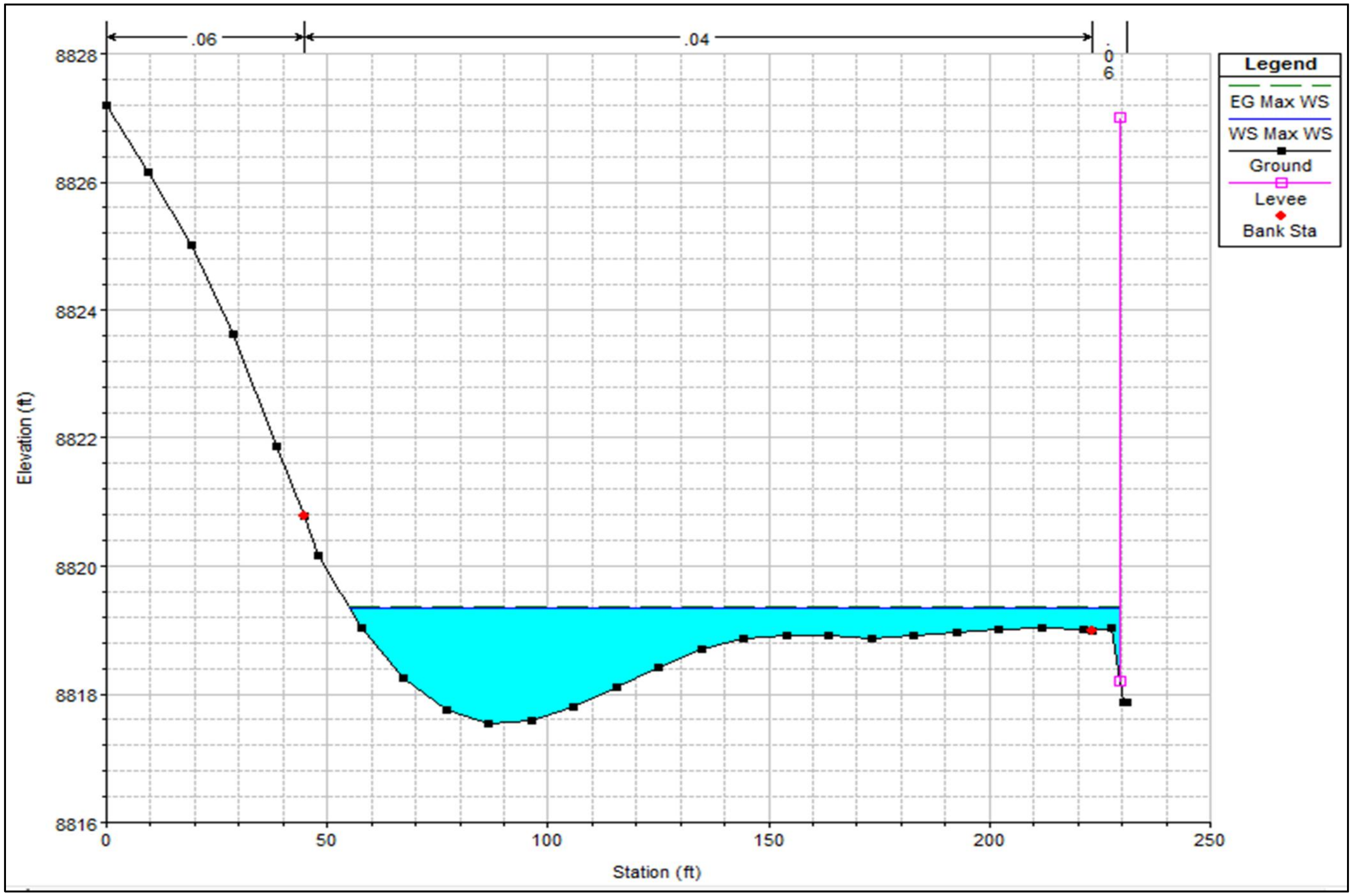


Figure 7.5C: Scenario 3 - Pond 15 Cross-Section 131.1

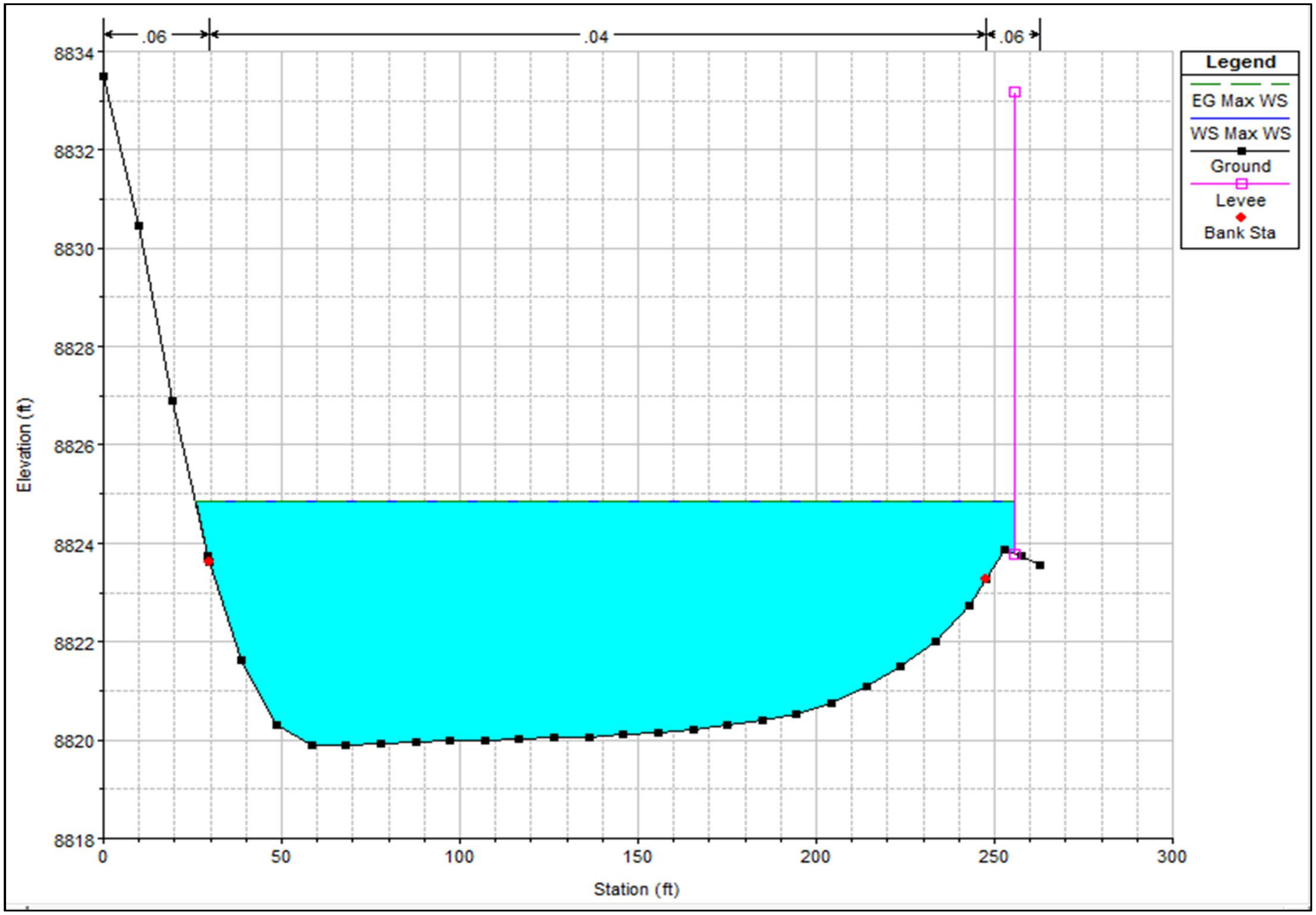


Figure 7.6A: Scenario 1 - Pond 18 Cross-Section 341

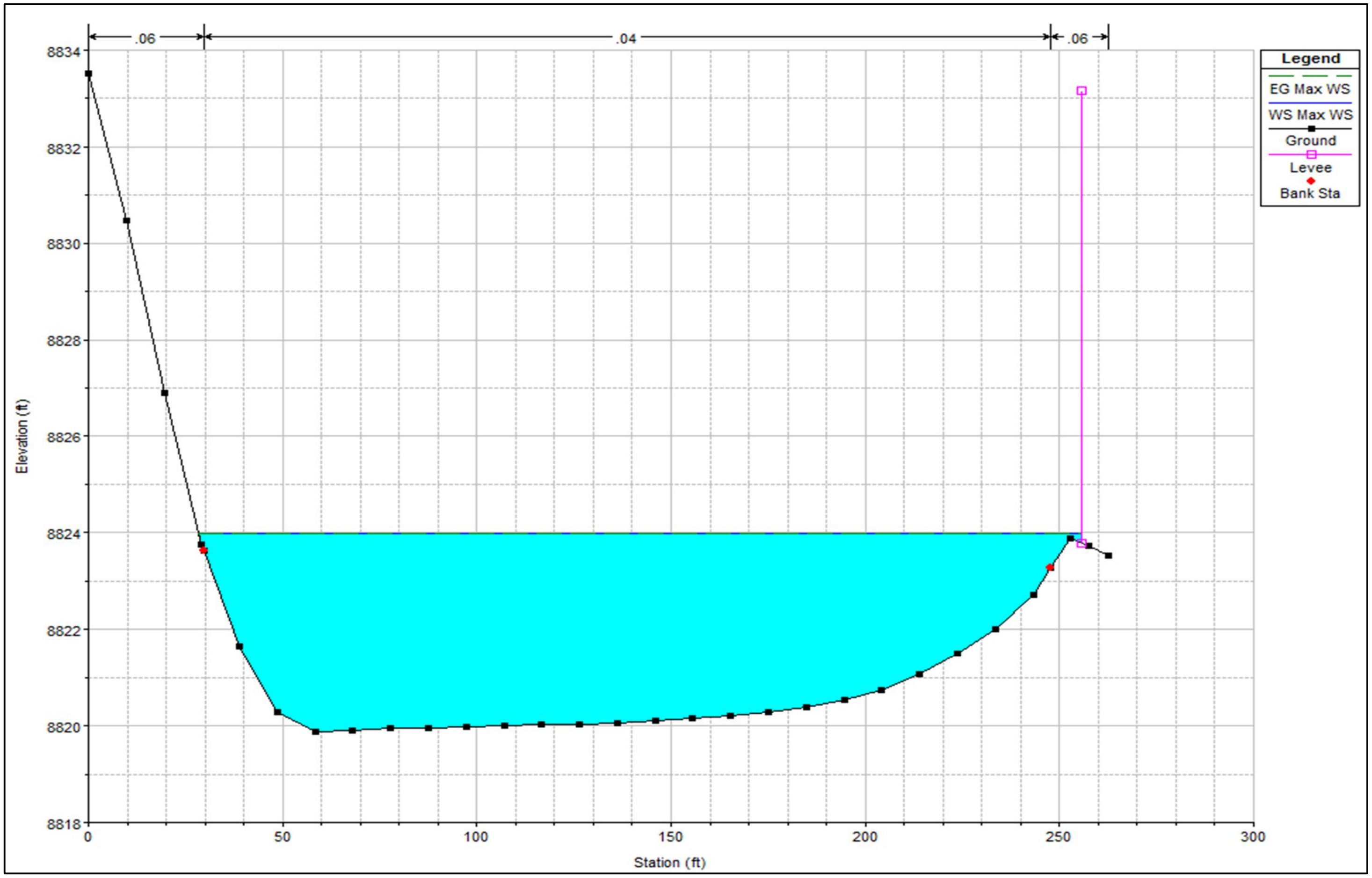


Figure 7.6B: Scenario 2 - Pond 18 Cross-Section 341

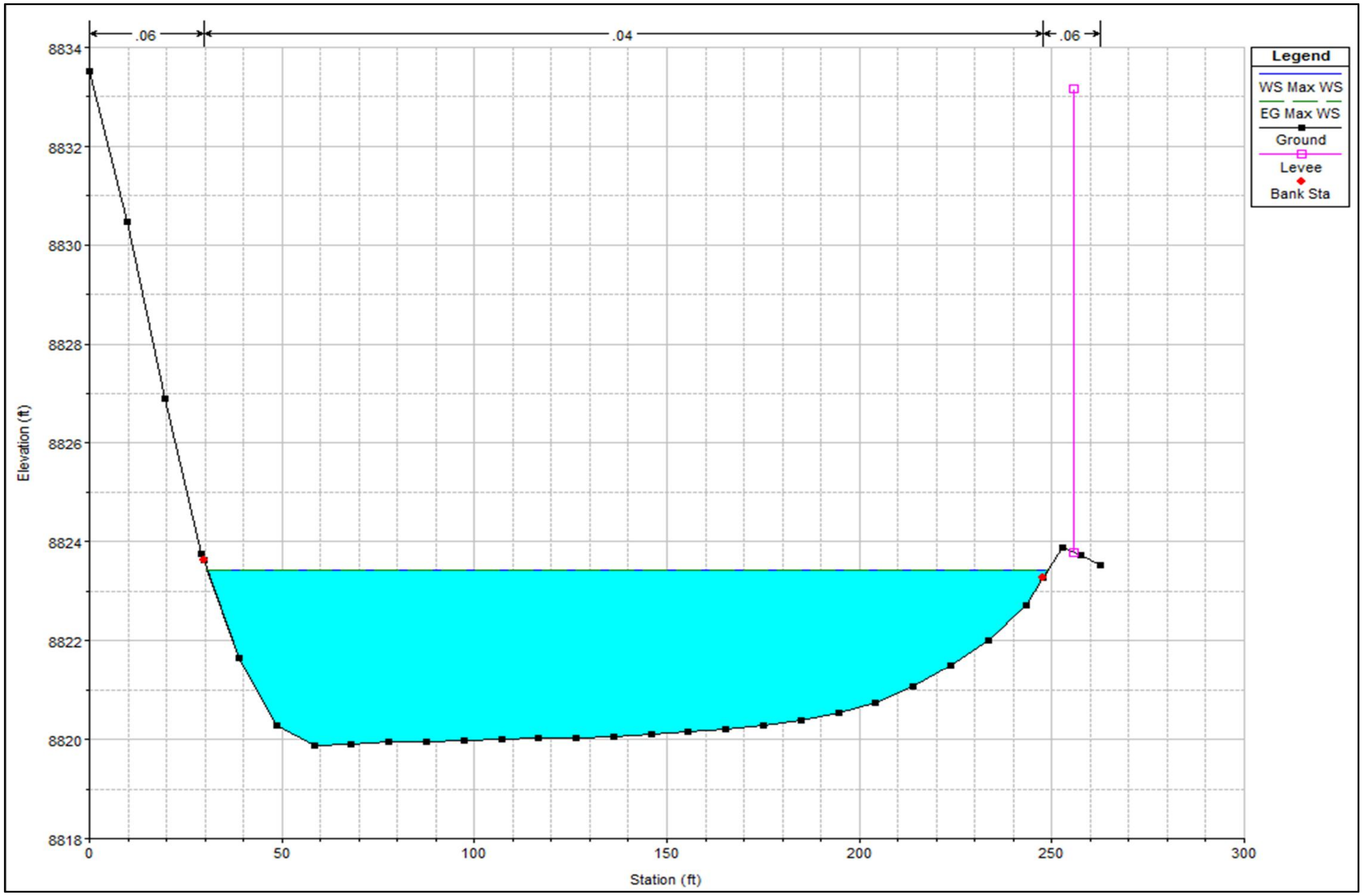


Figure 7.6C: Scenario 3 - Pond 18 Cross-Section 341

Design Sheets

RICO-ARGENTINE SITE-OU01

ADIT HYDRAULIC CONTROL
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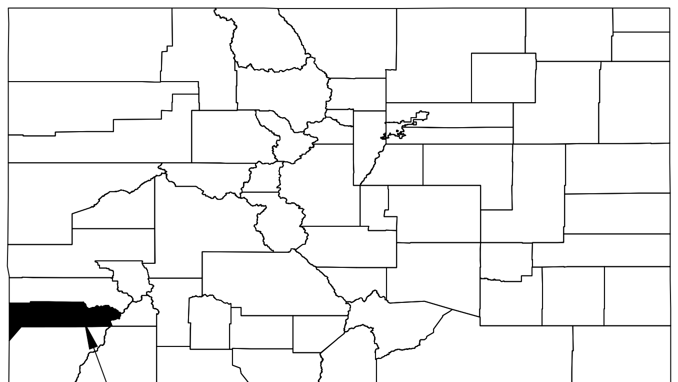
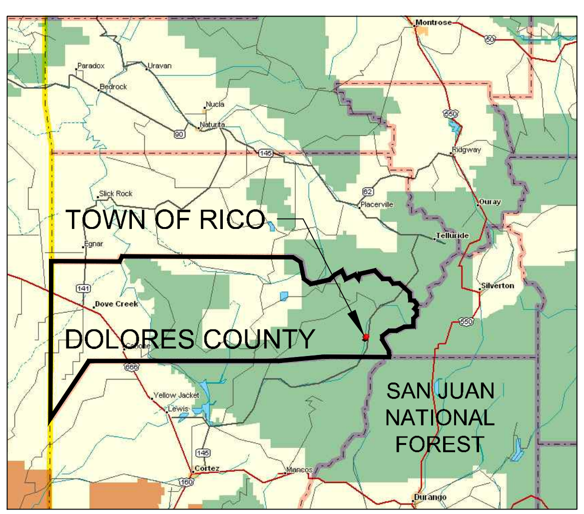
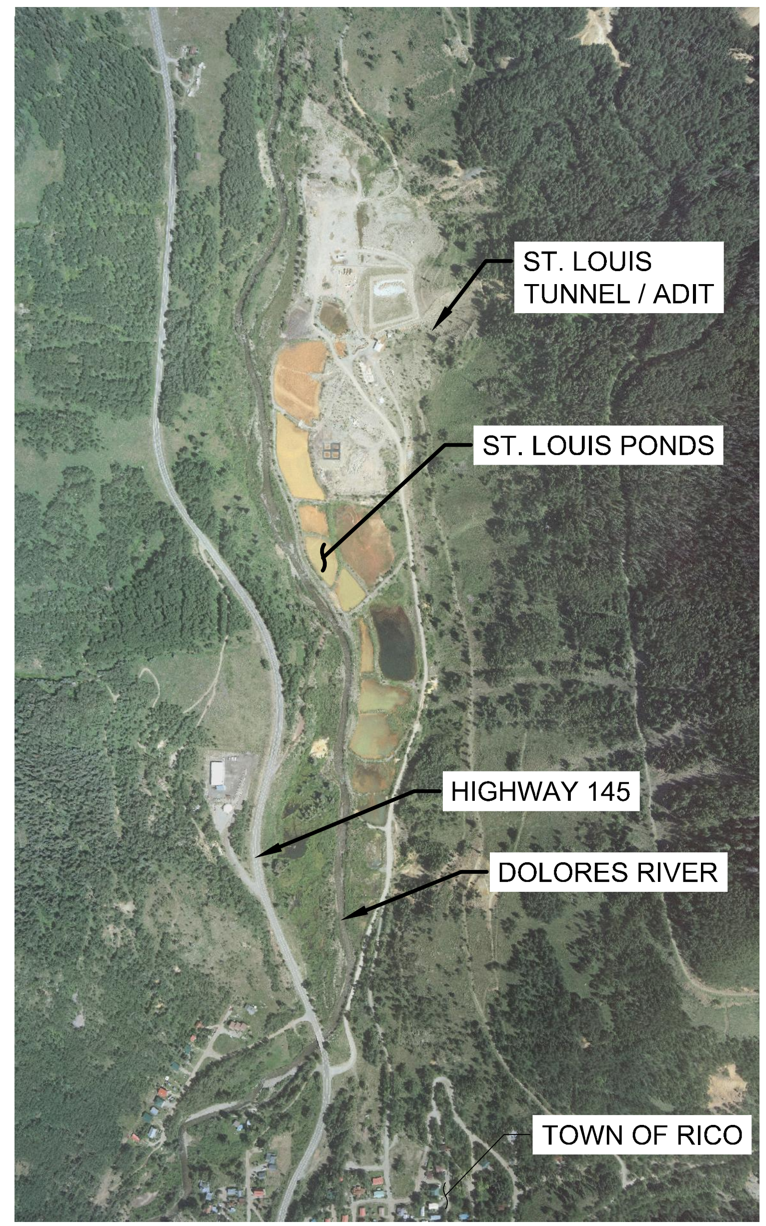


STATE OF COLORADO
NOT TO SCALE



DOLORES COUNTY
NOT TO SCALE



VICINITY MAP
SCALE: 1" = 500'

0 500'

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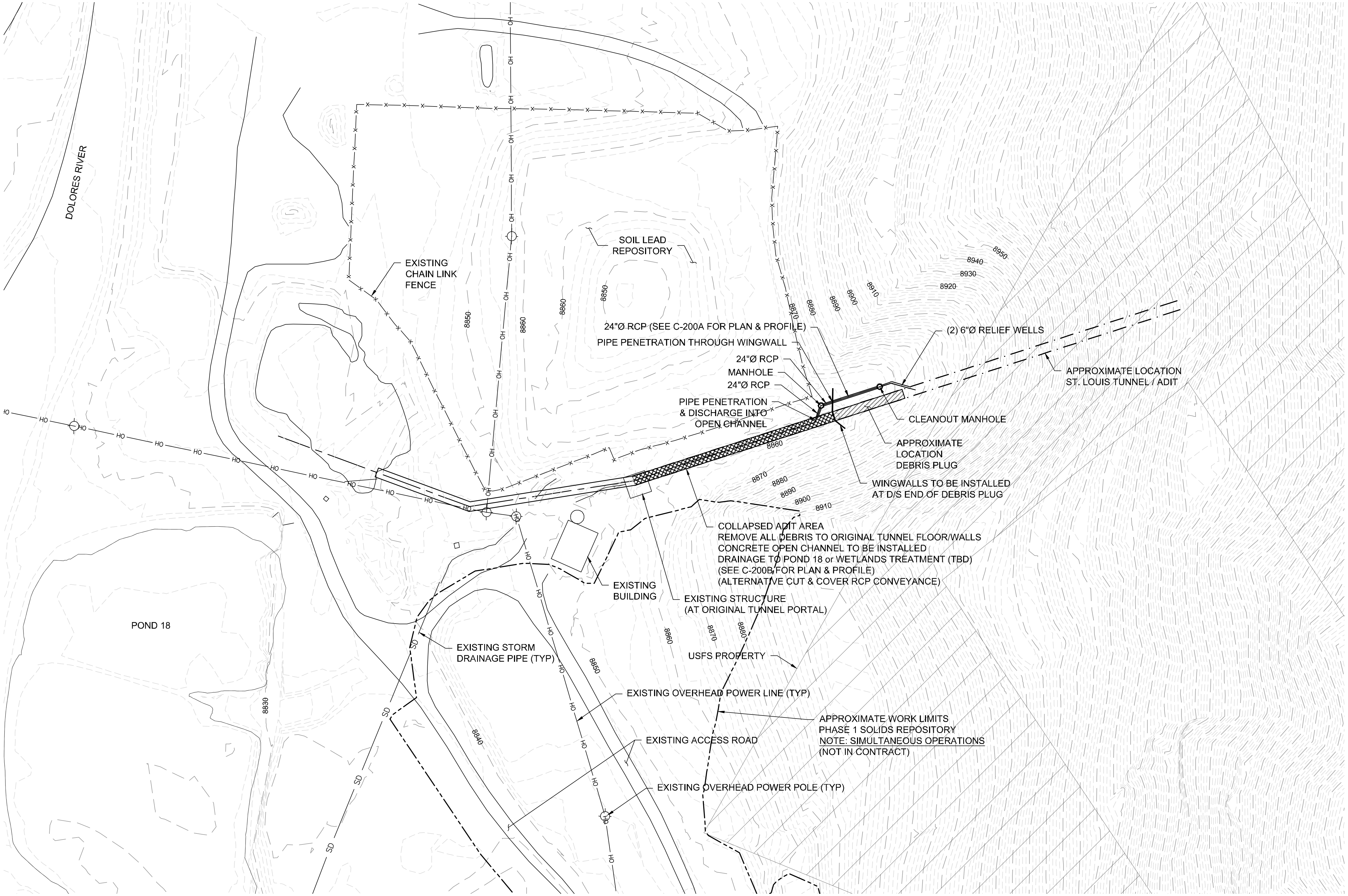
60306687

RICO-ARGENTINE SITE-OU01
ADIT HYDRAULIC CONTROL
ALTERNATIVE #2 - RELIEF WELLS

PRELIMINARY 30% DESIGN - OCTOBER 30, 2013

ANSI D 22" x 34"   Approved: DY   Checked: MQ   Designer: ACR   Project Management Initials: Last Plotted: 2013-10-21   Last saved by: RODDENA
Filename: Z:_CURRENT_PROJECTS\ATLANTIC RICHFIELD\60157757 RICO\000_CADD\02-SHEETS\01-ADIT-HYDRAULIC-CONTROL-30%\ALT-2_RELIEF-WELLS\G\60306687-SHT-20-0000-G-G001-CVR_AHC-ALT-2_30%.DWG



30% DESIGN
NOT FOR CONST

OVERALL SITE & STAGING PLAN
1" = 50'
(on ANSI D 22"x34")

0 50' 100' 200'

N

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PROJECT
RICO-ARGENTINE SITE-OU01

ADIT HYDRAULIC CONTROL

ALTERNATIVE #2 RELIEF WELLS

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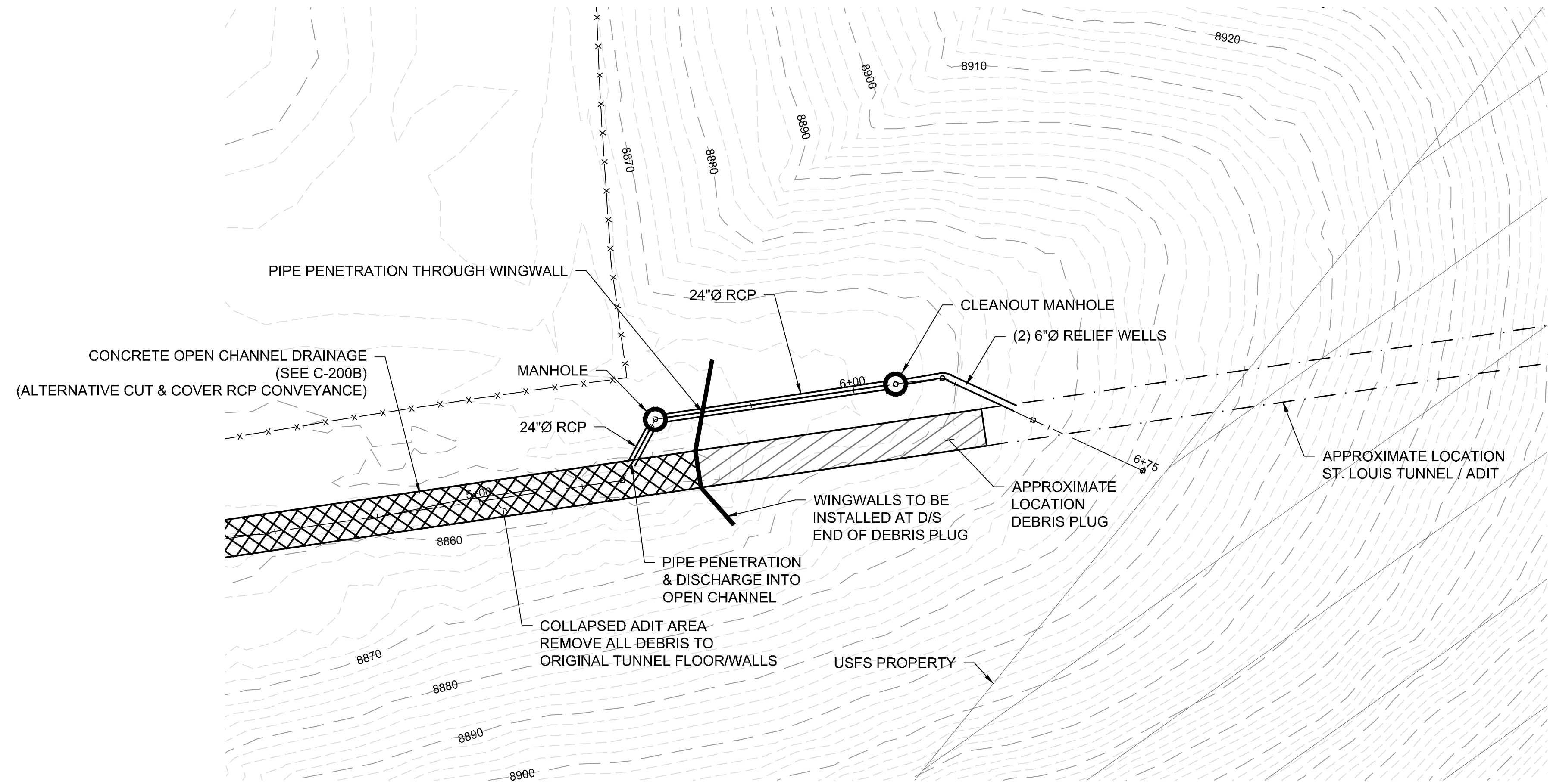
60306687

SHEET TITLE
OVERALL SITE & STAGING PLAN

SHEET NUMBER
C-100 2 OF 5



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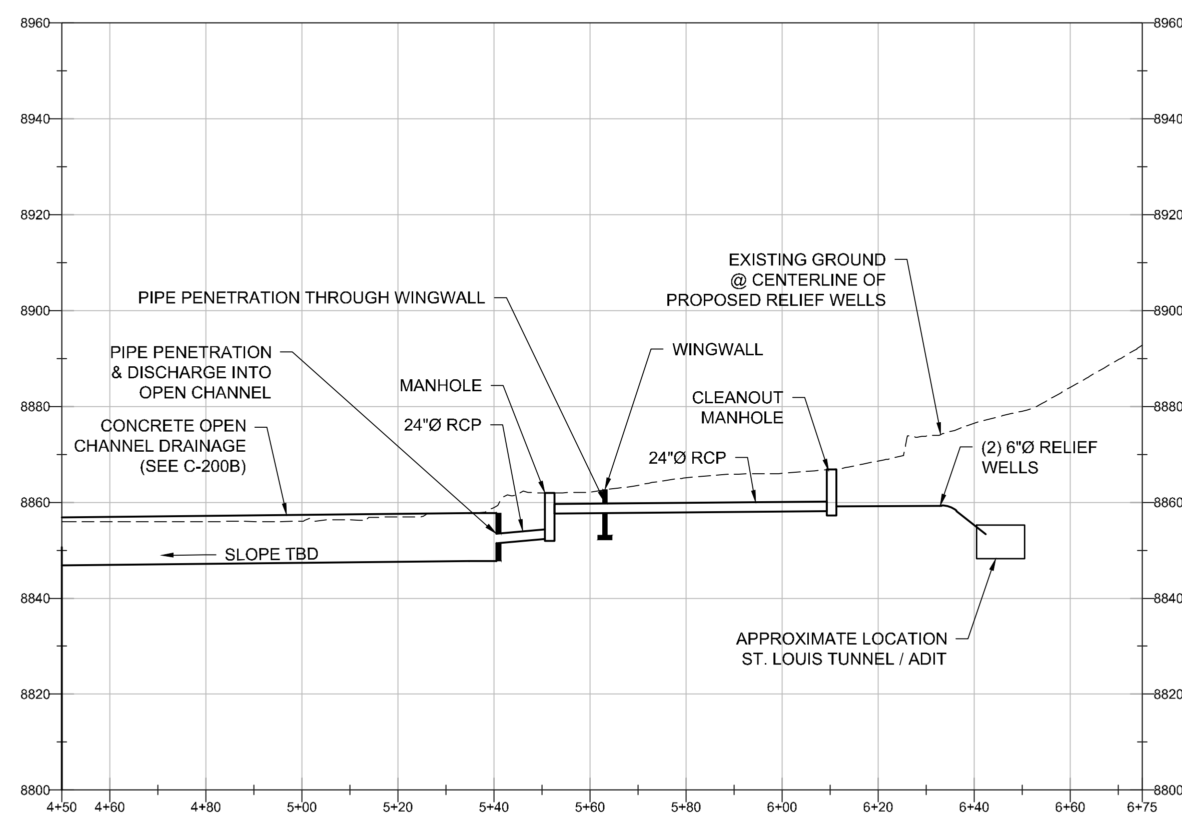

PROFILE (OPEN CHANNEL)

1" = 20'
(on ANSI D 22"x34")

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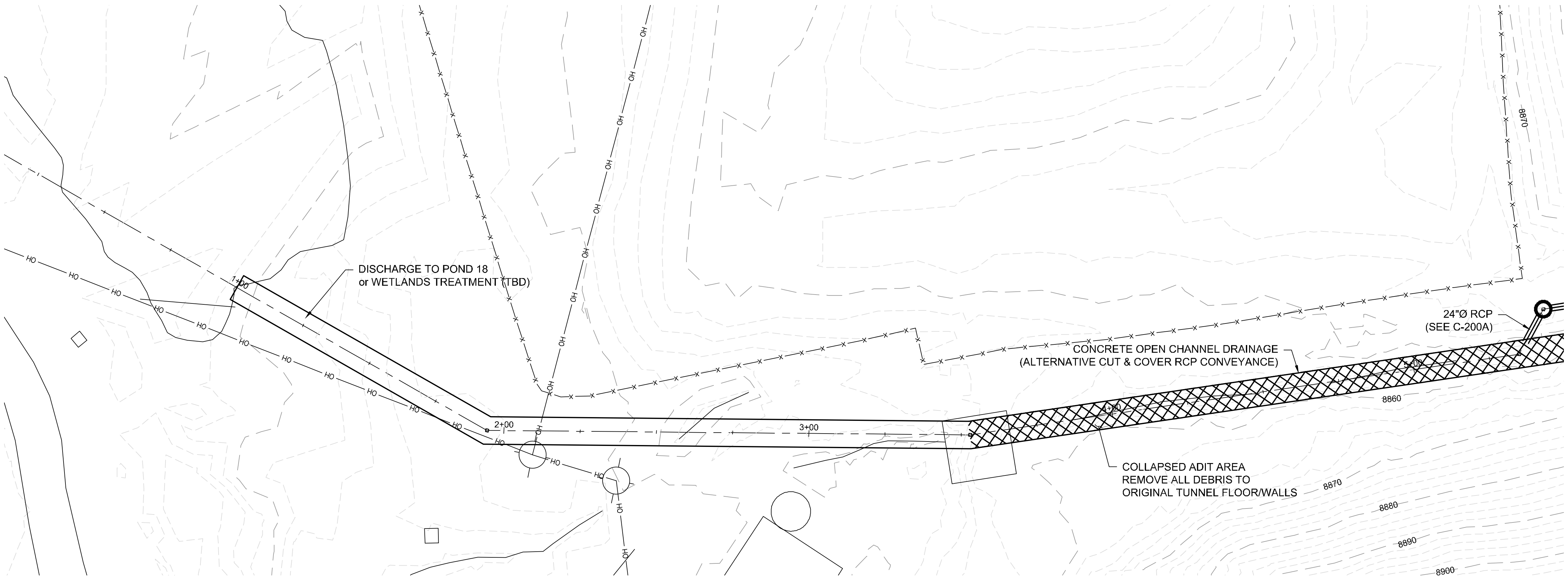
SHEET NUMBER

C-200A

ANSI D 22" x 34"



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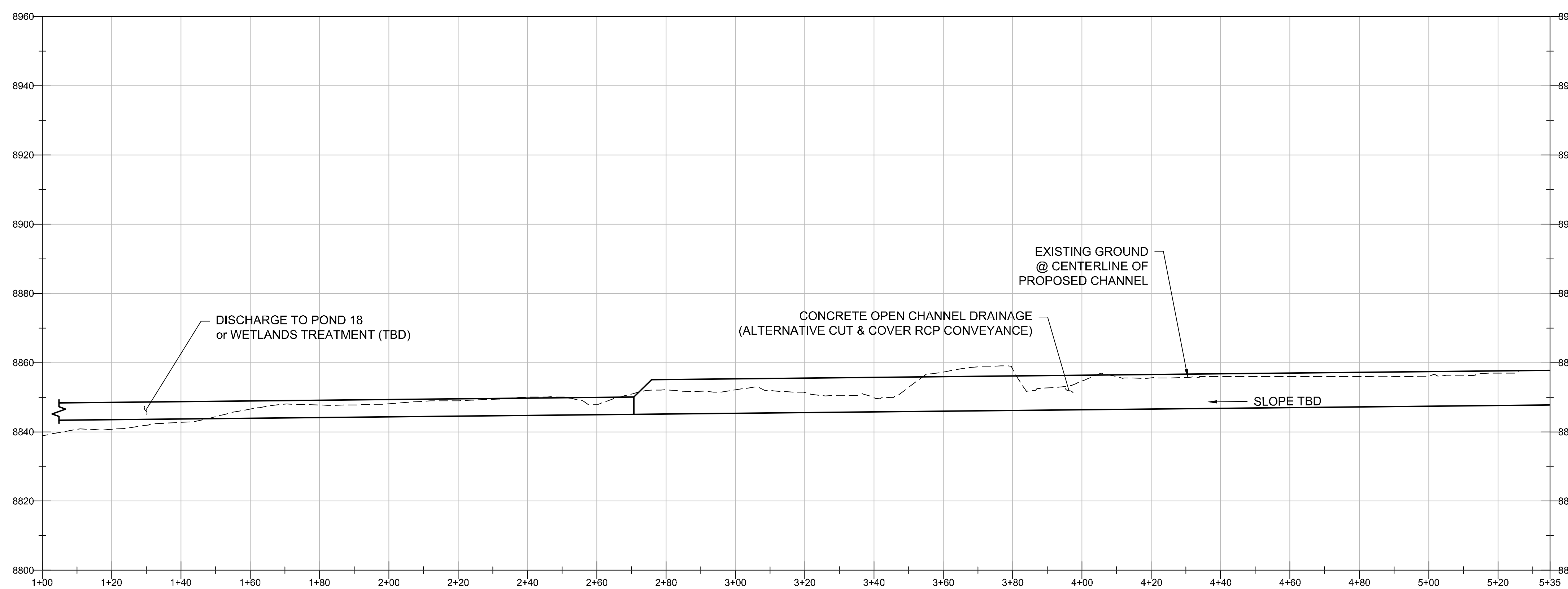

PROFILE (OPEN CHANNEL)
1" = 20'
(on ANSI D 22"x34")

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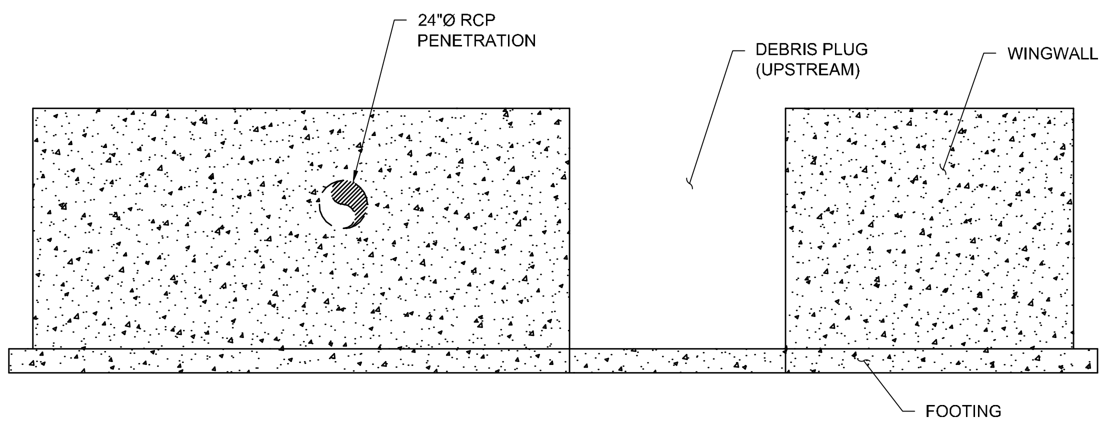
C-200B 4 OF 5



WINGWALL DETAIL
(LOOKING UPSTREAM)
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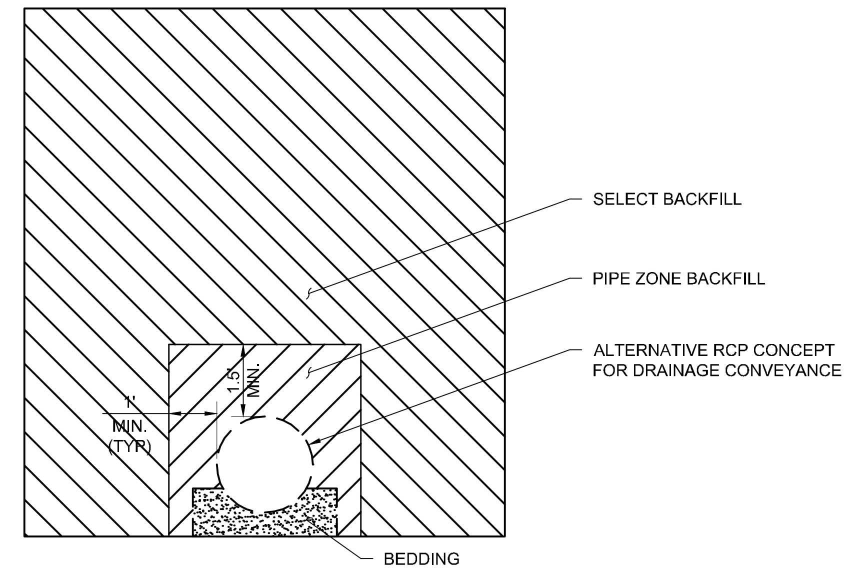
(on ANSI D 22"x34")



ALTERNATIVE CUT & COVER RCP SECTION
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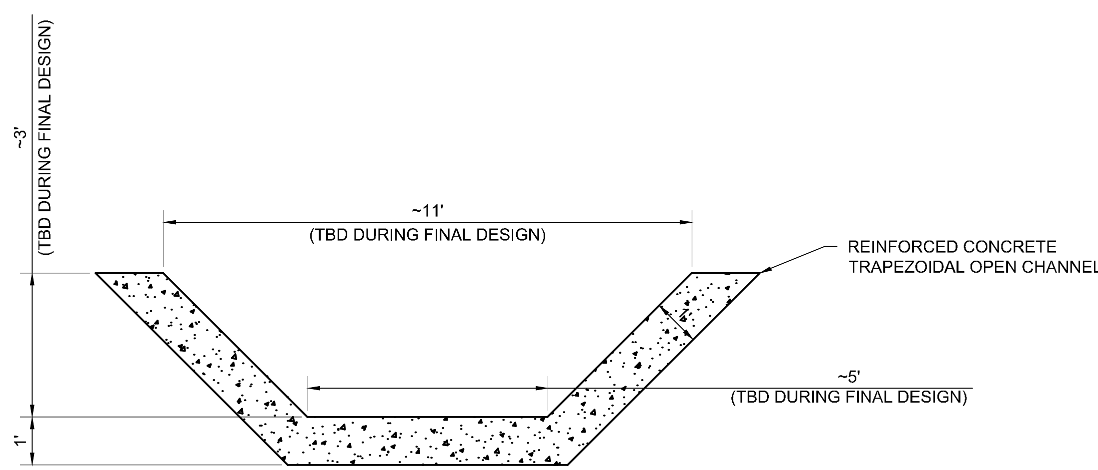
(on ANSI D 22"x34")



TYPICAL TRAPEZOIDAL OPEN CHANNEL SECTION
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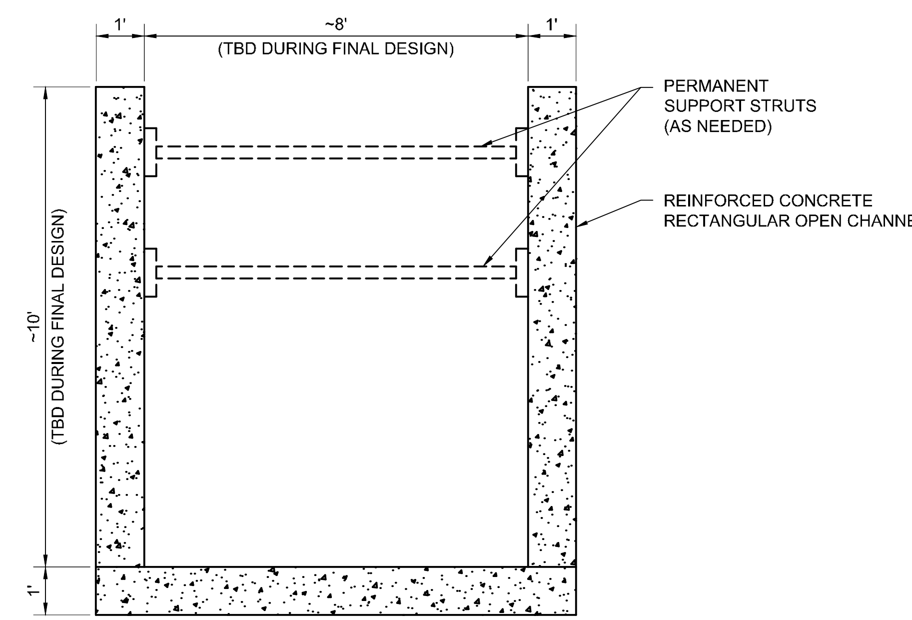
(on ANSI D 22"x34")



TYPICAL RECTANGULAR OPEN CHANNEL SECTION
1/2" = 1'-0"
(on ANSI D 22"x34")

NOTES:
1. ALL DIMENSIONS CONCEPTUAL, SUBJECT TO CHANGE.
2. REINFORCING NOT SHOWN, TBD FOR FINAL DESIGN.

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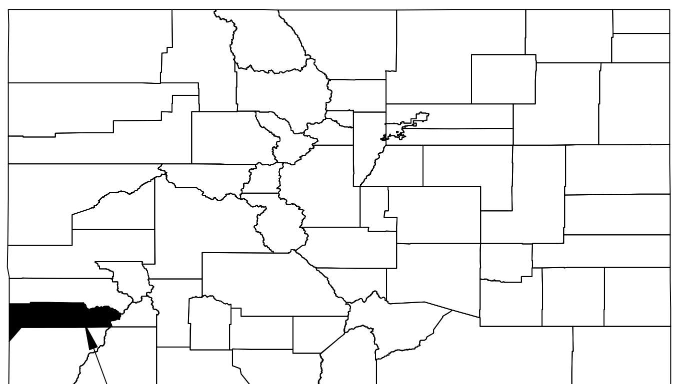
SECTIONS & DETAILS (OPEN CHANNEL)

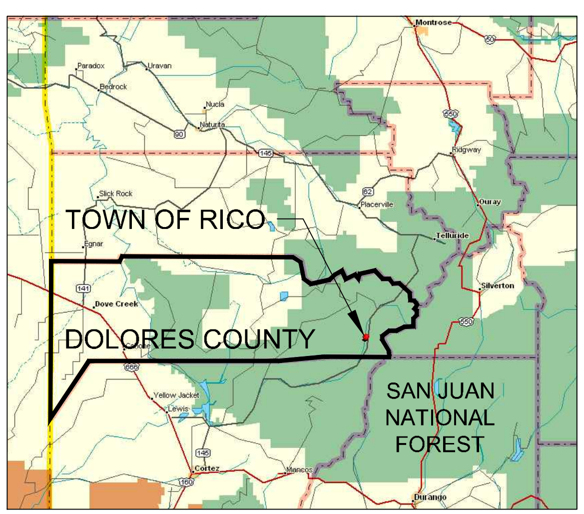
SHEET NUMBER
C-310 5 OF 5

RICO-ARGENTINE SITE-OU01

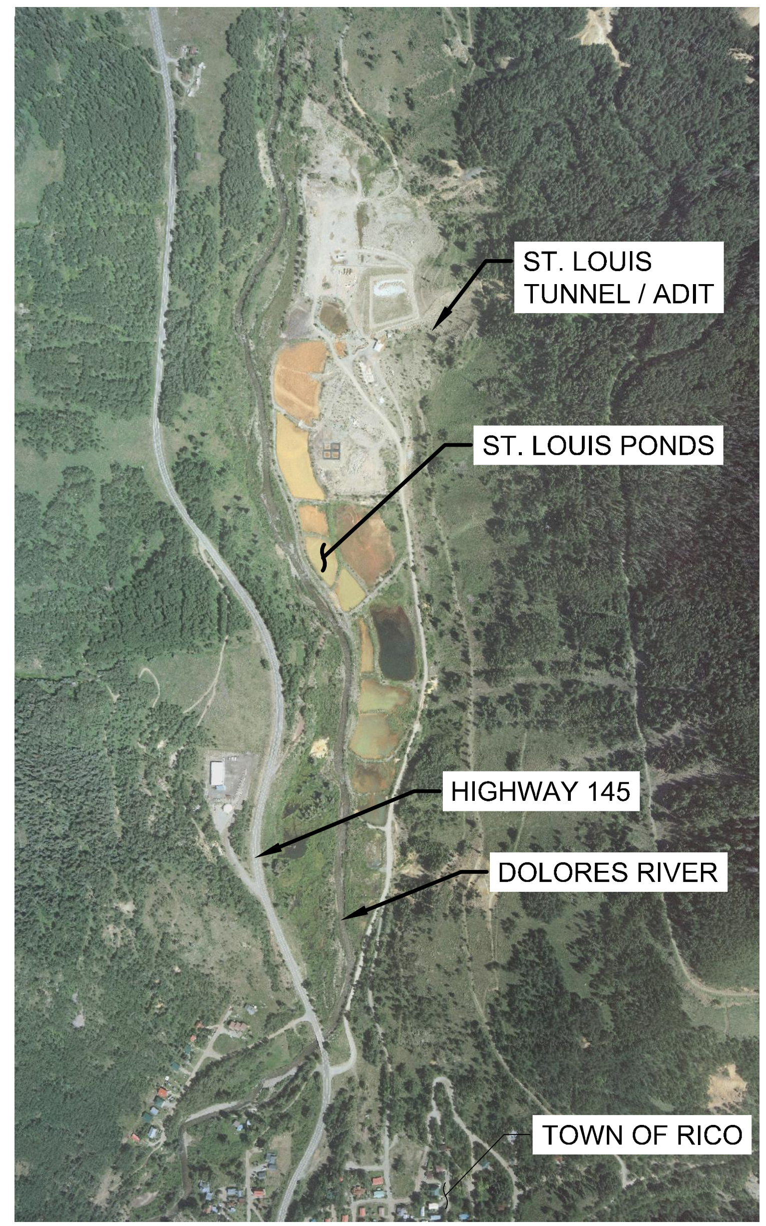
ADIT HYDRAULIC CONTROL
ALTERNATIVE #5 - TUNNELING



STATE OF COLORADO
NOT TO SCALE



DOLORES COUNTY
NOT TO SCALE



VICINITY MAP
SCALE: 1" = 500'

0 500'

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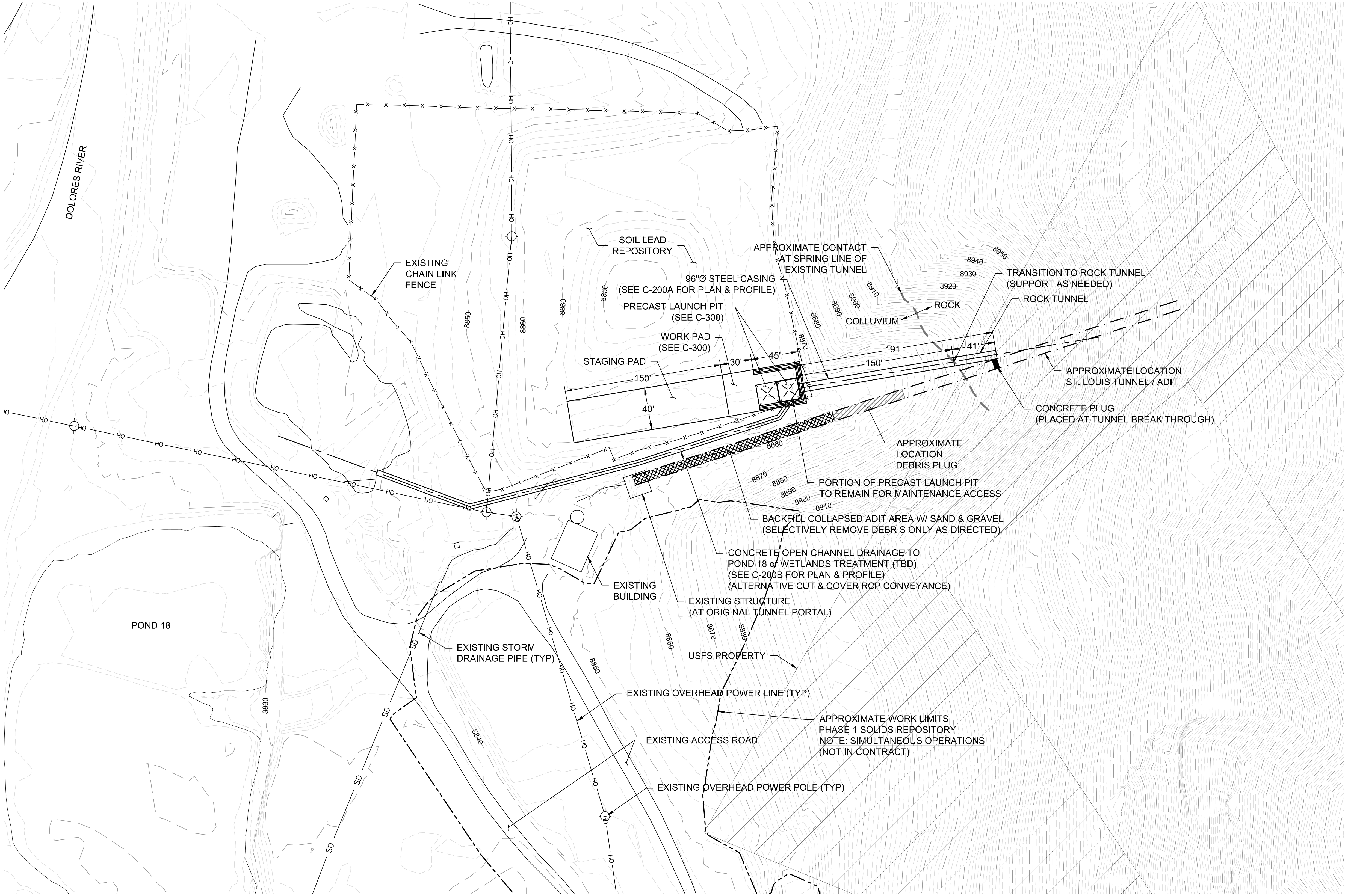
RICO-ARGENTINE SITE-OU01
ADIT HYDRAULIC CONTROL
ALTERNATIVE #5 - TUNNELING

PRELIMINARY 30% DESIGN - OCTOBER 30, 2013

Filename: Z:_CURRENT_PROJECTS\ATLANTIC RICHFIELD\60157757 RICO000_CADD\02-SHEETS\01-ADIT-HYDRAULIC-CONTROL-30%\ALT-5_TUNNELING\G\60306687-SHT-20-0000-G-G001-CVR_AHC-ALT-5_30%.DWG Last saved by: RODDENA Last Plotted: 2013-10-21 Project Management Initials: Designer: ACR Checked: MQ Approved: DY ANSI D 22" x 34"



30% DESIGN
NOT FOR CONST

OVERALL SITE & STAGING PLAN
1" = 50'
(on ANSI D 22"x34")

0 50' 100' 200'

N

ANSI D 22" x 34"

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C-100 2 OF 6

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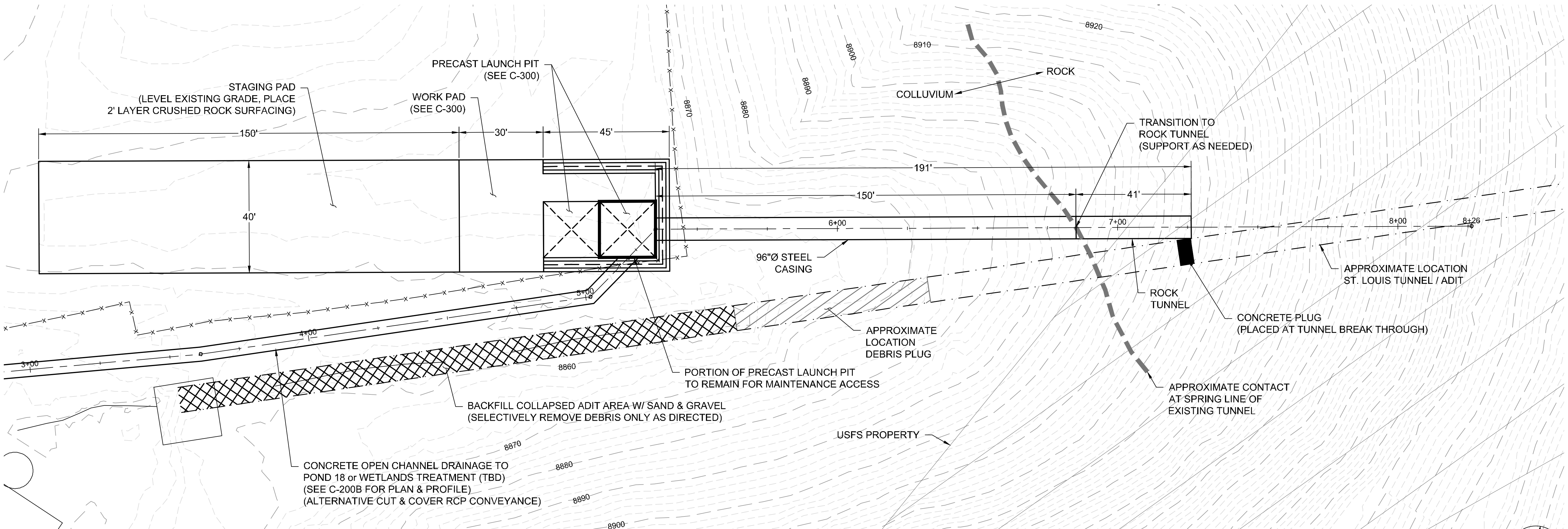

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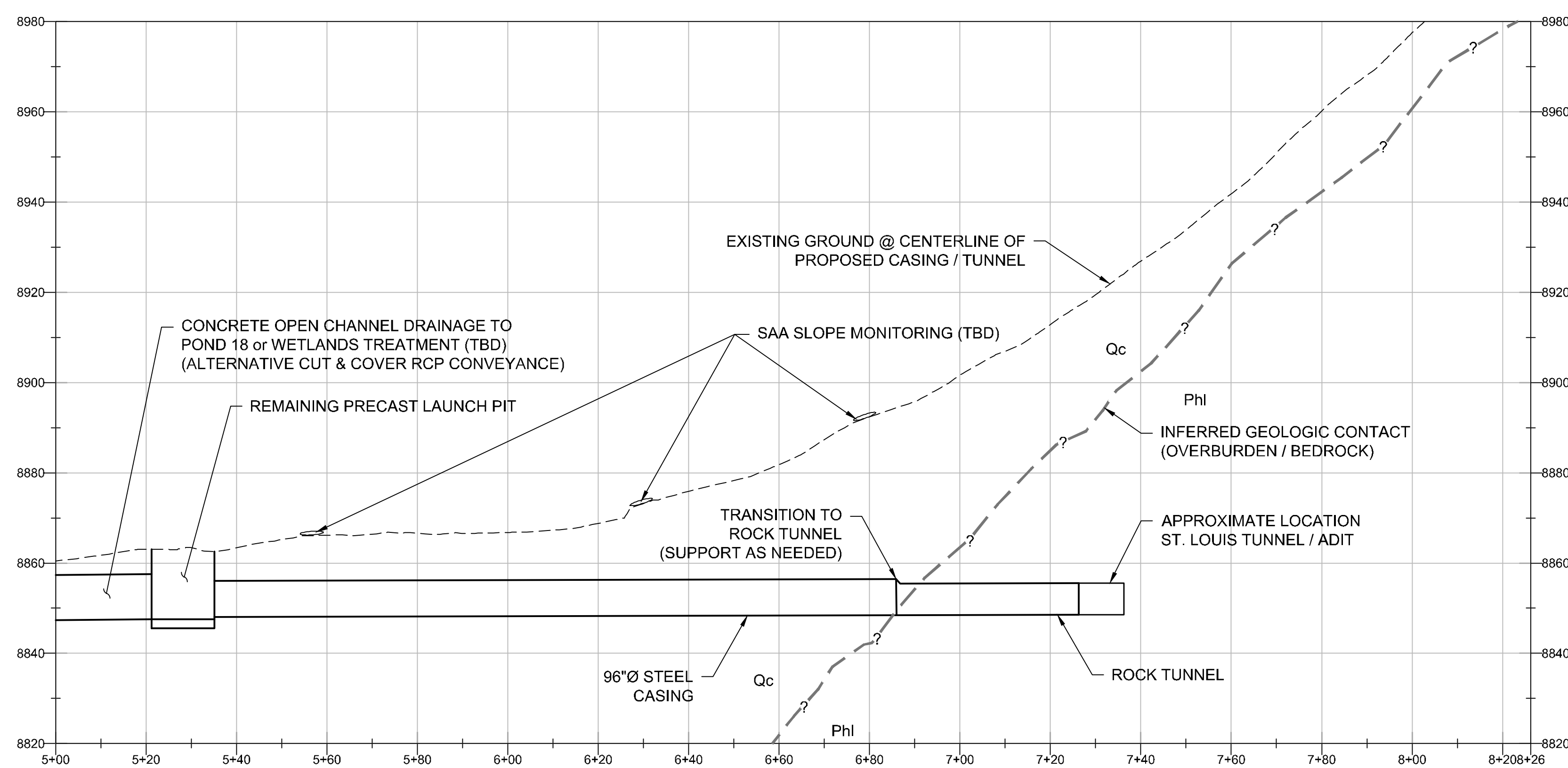

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PROFILE (TUNNELING)

1" = 20'
(on ANSI D 22"x34")

0 20' 40' 80'

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SHEET NUMBER

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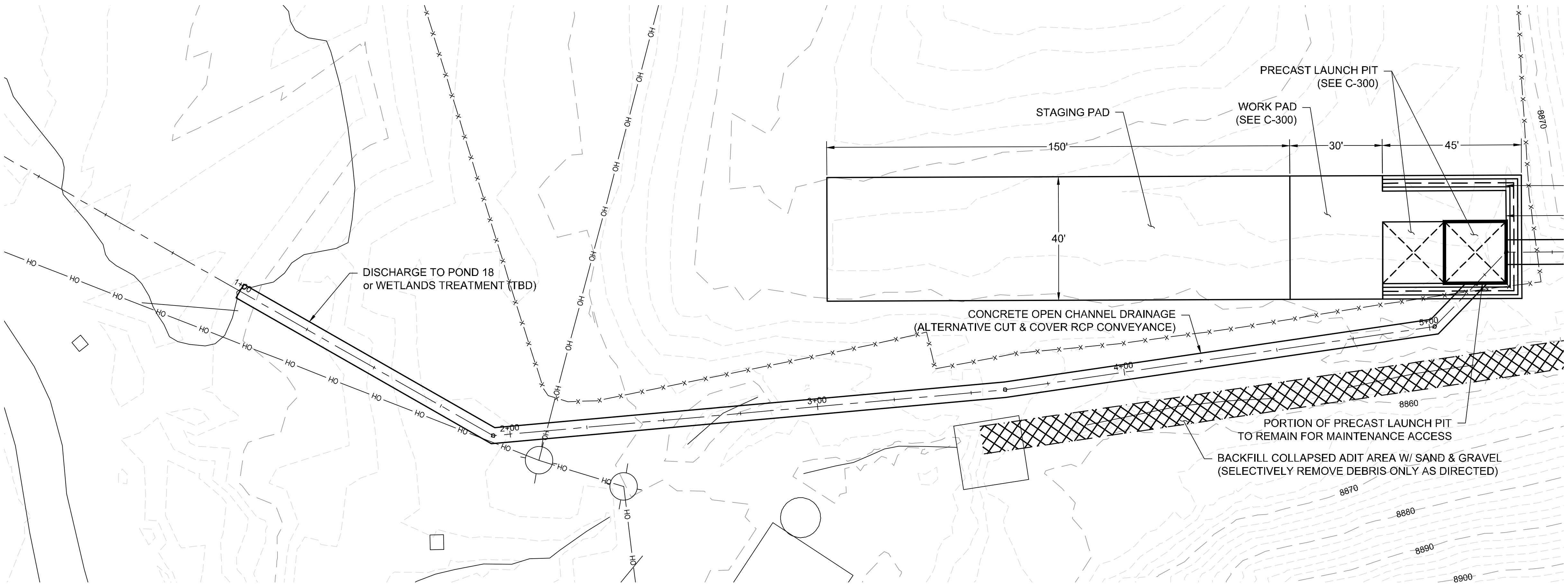
ANSI D 22" x 34"



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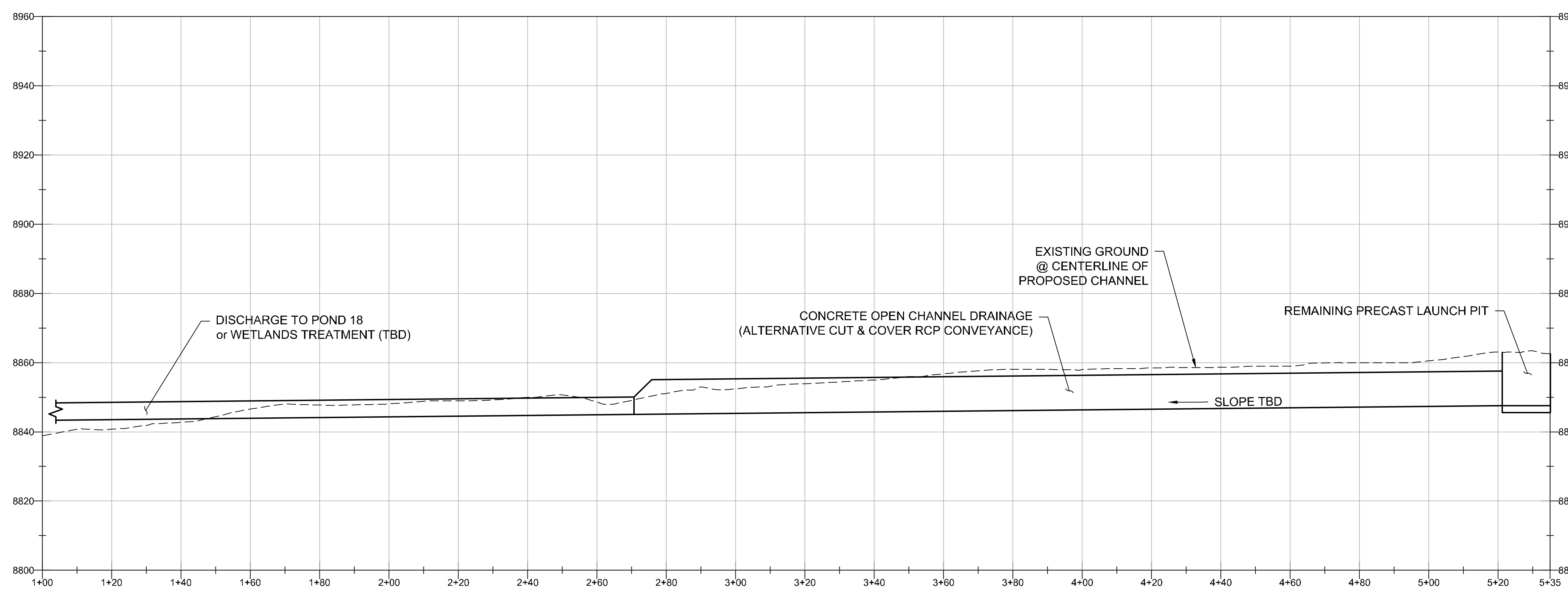

PROFILE (OPEN CHANNEL)

1" = 20'
(on ANSI D 22"x34")

0 20' 40' 80'

30% DESIGN
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PLAN & PROFILE (OPEN CHANNEL)

SHEET NUMBER

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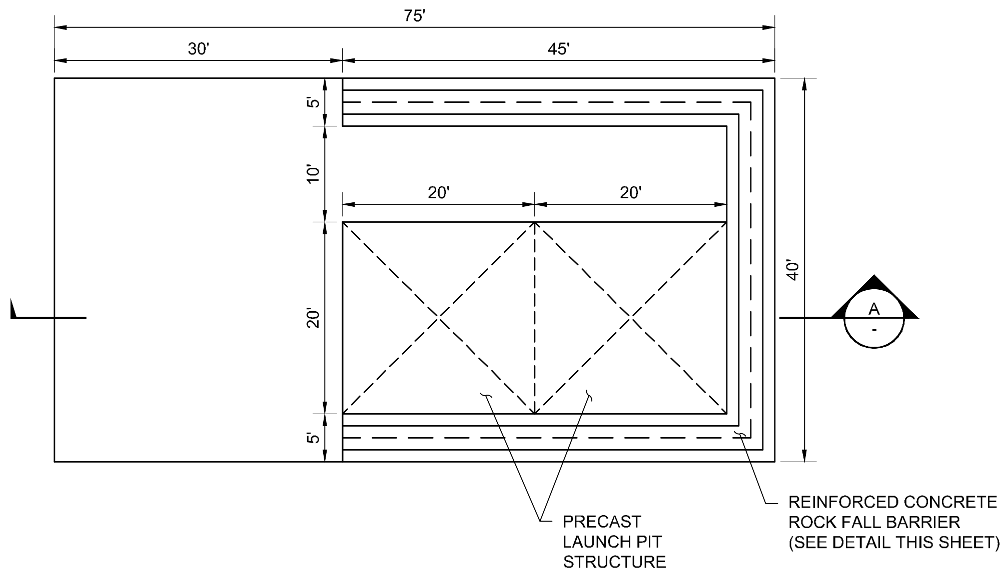
ANSI D 22" x 34"



WORK PAD PLAN

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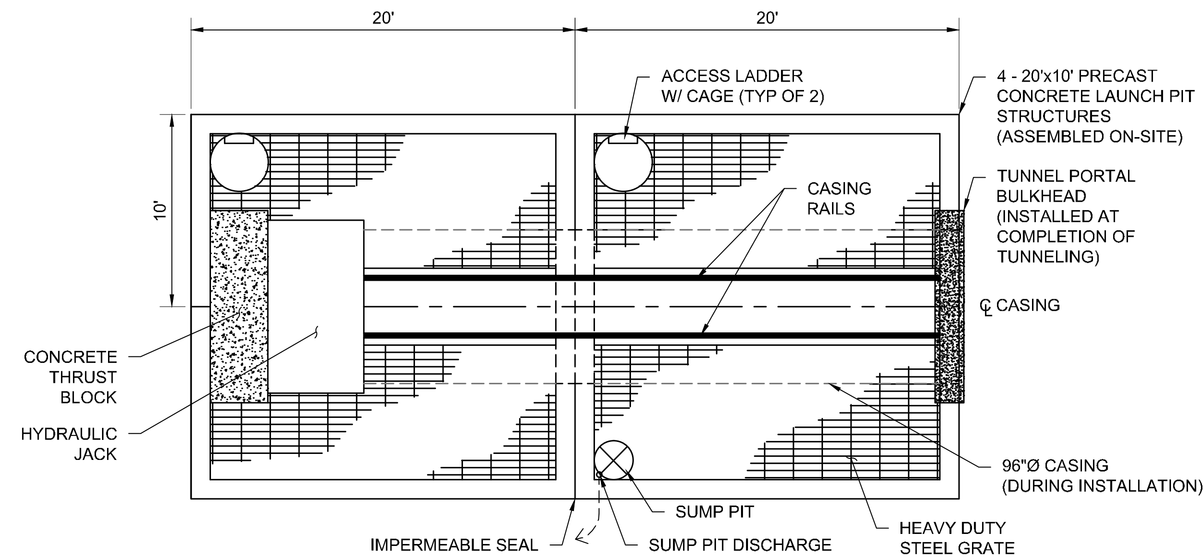

PRECAST LAUNCH PIT PLAN

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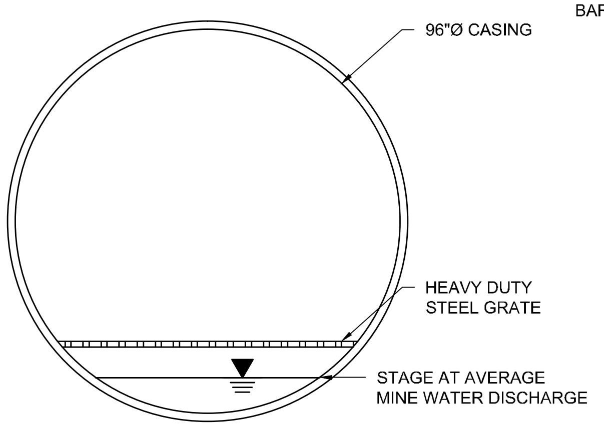

TYPICAL CASING SECTION

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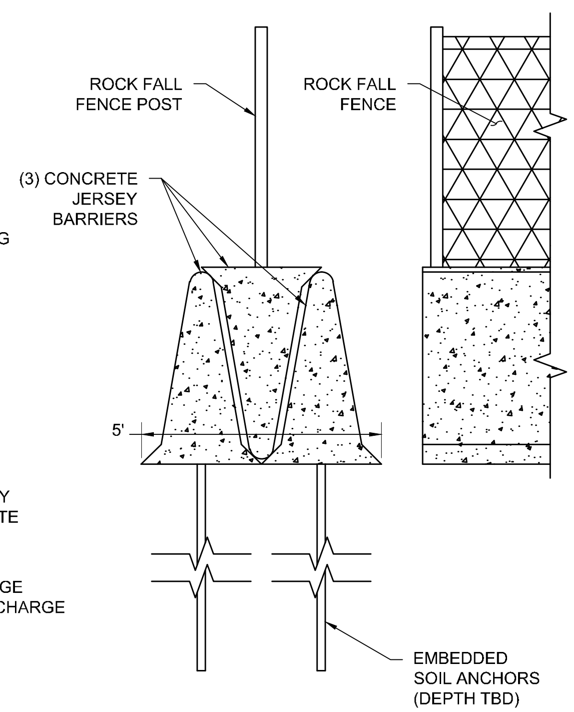

REINFORCED CONCRETE ROCK FALL BARRIER DETAIL

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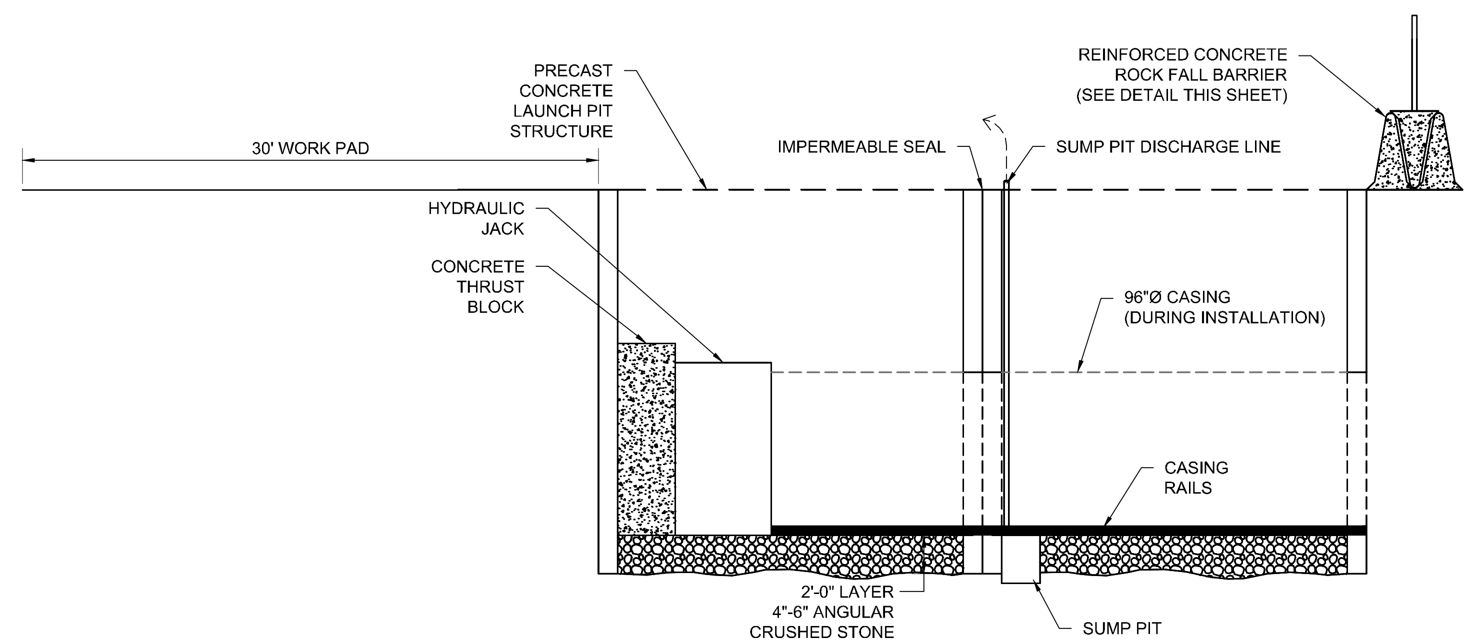

A PRECAST LAUNCH PIT PROFILE

1" = 5' (on ANSI D 22"x34")

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PROJECT

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ALTERNATIVE #5 TUNNELING

CLIENT

ATLANTIC RICHFIELD COMPANY

CONSULTANT

AECOM
717 17th STREET
SUITE 2600
DENVER, CO 80202
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www.aecom.com

CONSULTANTS

ANDERSON ENGINEERING COMPANY, INC.

REGISTRATION

ISSUE/REVISION

I/R	DATE	DESCRIPTION

PROJECT NUMBER

60306687

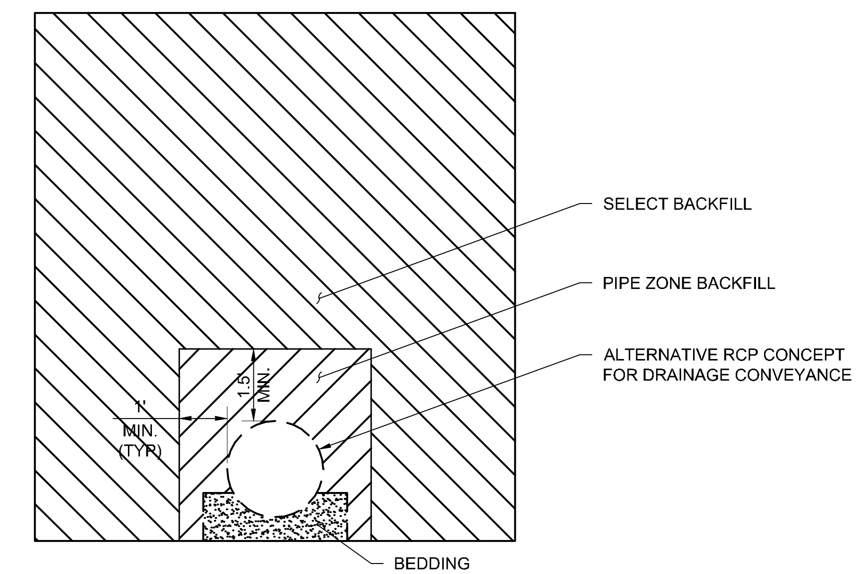
SHEET TITLE

SECTIONS & DETAILS (TUNNELING)

SHEET NUMBER

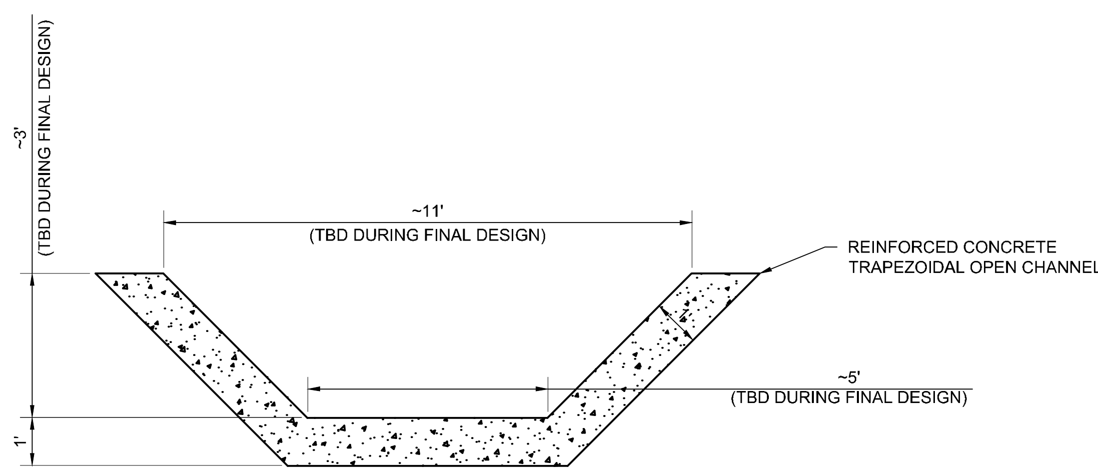
C-300 5 OF 6

ANSI D 22" x 34"



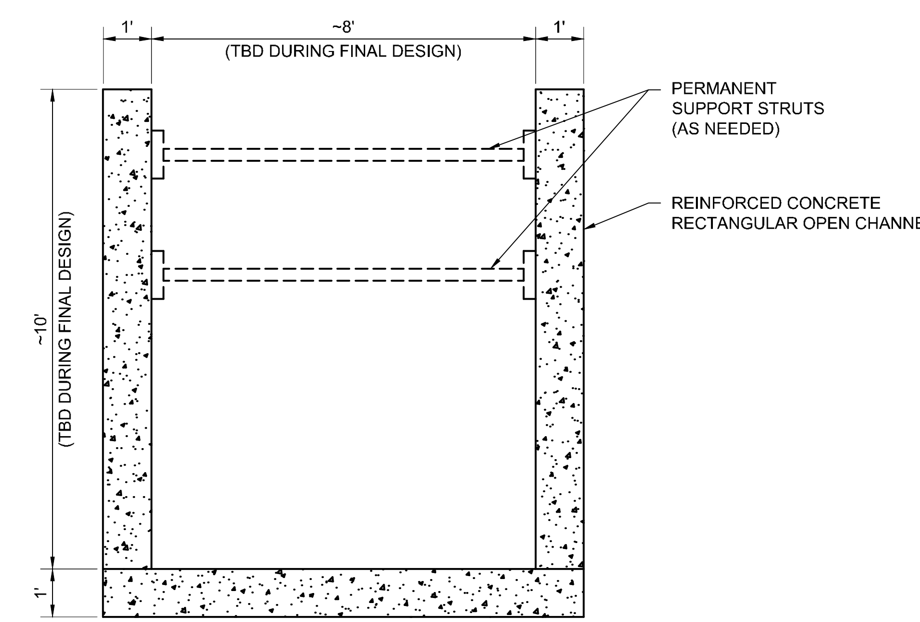
ALTERNATIVE CUT & COVER RCP SECTION

1/2" = 1'-0"
(on ANSI D 22"x34")



TYPICAL TRAPEZOIDAL OPEN CHANNEL SECTION

1/2" = 1'-0"
(on ANSI D 22"x34")



TYPICAL RECTANGULAR OPEN CHANNEL SECTION

1/2" = 1'-0"
(on ANSI D 22"x34")

NOTES:
1. ALL DIMENSIONS CONCEPTUAL, SUBJECT TO CHANGE.
2. REINFORCING NOT SHOWN, TBD FOR FINAL DESIGN.

AECOM

PROJECT

RICO-ARGENTINE
SITE-OU01

ADIT HYDRAULIC
CONTROL

ALTERNATIVE #5
TUNNELING

CLIENT

ATLANTIC
RICHFIELD
COMPANY

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ENGINEERING
COMPANY, INC.

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ISSUE/REVISION

I/R	DATE	DESCRIPTION

PROJECT NUMBER

60306687

SHEET TITLE

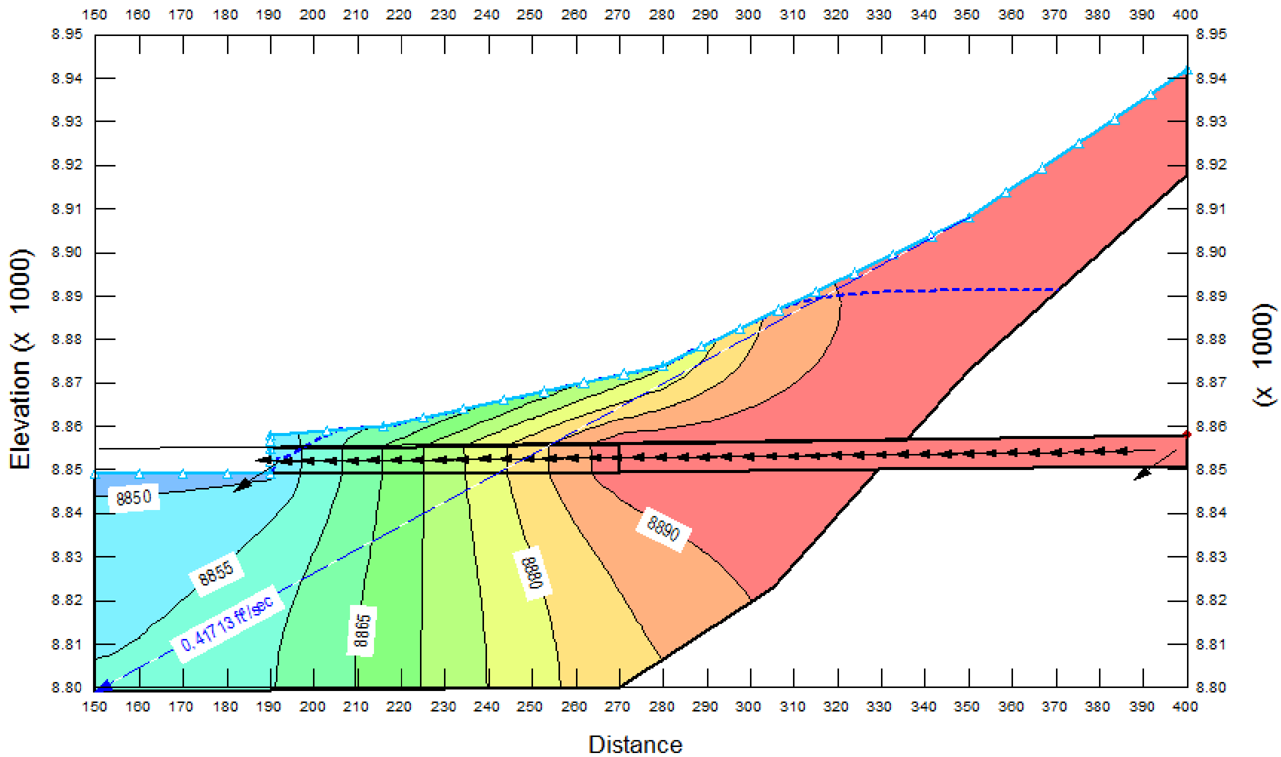
SECTIONS & DETAILS
(OPEN CHANNEL)

SHEET NUMBER

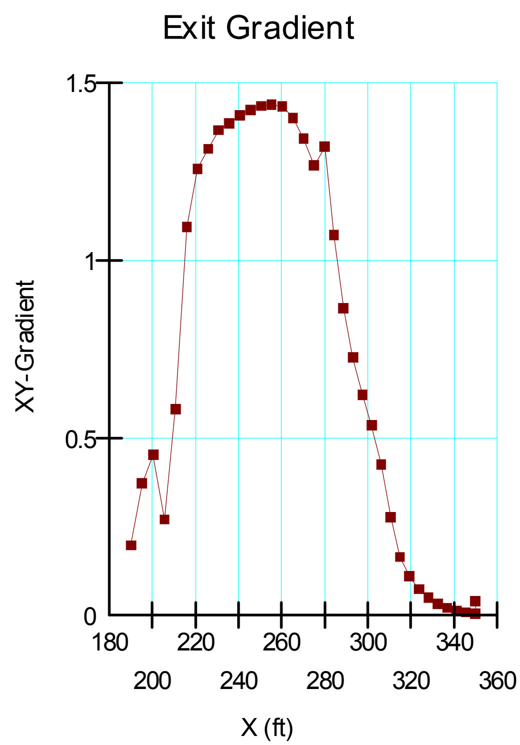
C-310 6 OF 6

Appendix A
Geotechnical Analyses

Debris Plug/Colluvium - Seepage and Stability



Seepage analysis results for tunnel head 8895 for Alternatives 0-2.



Exit gradient along the seepage face for tunnel elevation 8895 from seepage analysis for Alternatives 0-2.

Uplift Pressure (P)

Total Head A:
$H = 8895 - 8858 = 37\ ft$
Top Tunnel

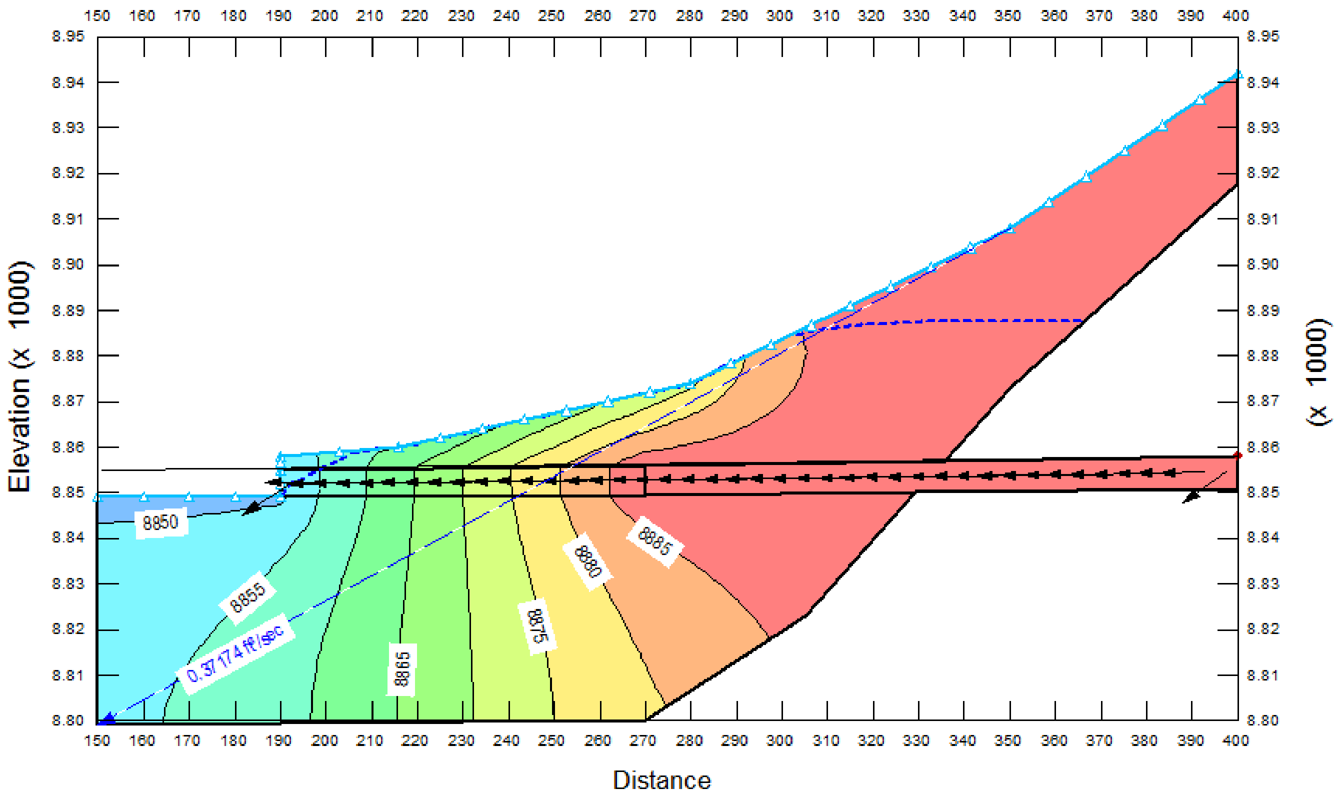
$P = 37\ ft\ x\ 62.4 = 2308\ ps$

Weight Resisting Soil

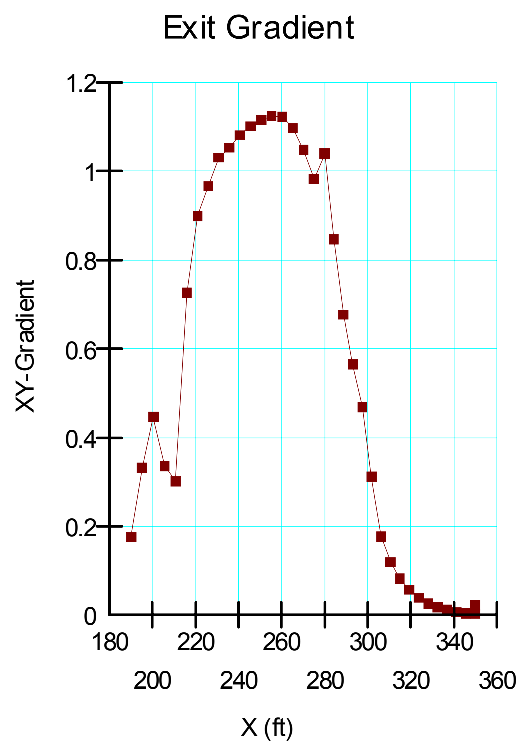
$W = (8873 - 8858)\gamma_T = 1875\ pcf$
$\gamma_T = 125\ pcf$

$$FS_{UPLIFT} = \frac{W}{P} = \frac{1875}{2308} = 0.81$$

Uplift Factor of Safety calculations for tunnel head 8895 for Alternatives 0-2.



Seepage analysis results for tunnel head 8890 for Alternatives 0-2.



Exit gradient along the seepage face for tunnel elevation 8890 from seepage analysis for Alternatives 0-2.

Uplift Pressure (P)

$H = 8890 - 8858 = 32\ ft$

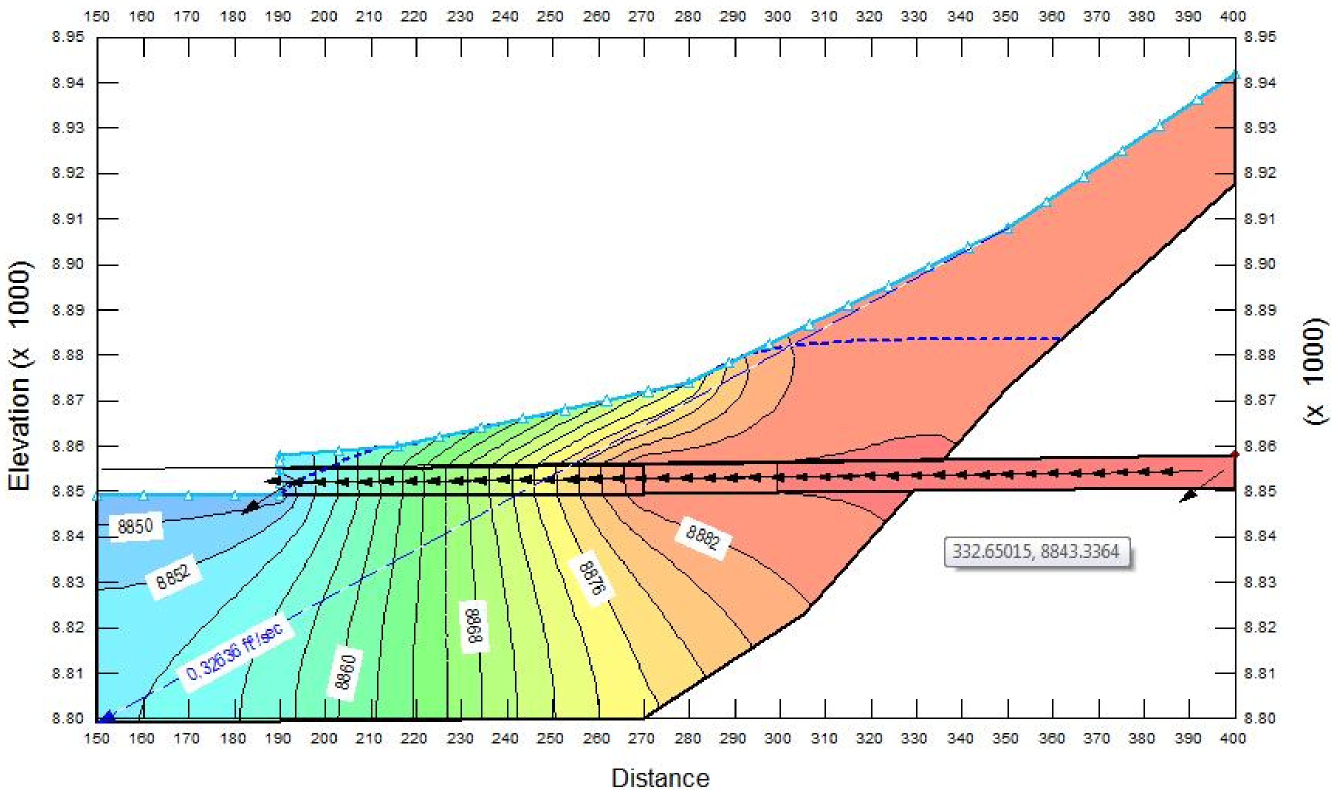
$P = 32\ ft\ x\ 62.4 = 1997\ psf$

Weight Resisting Soil

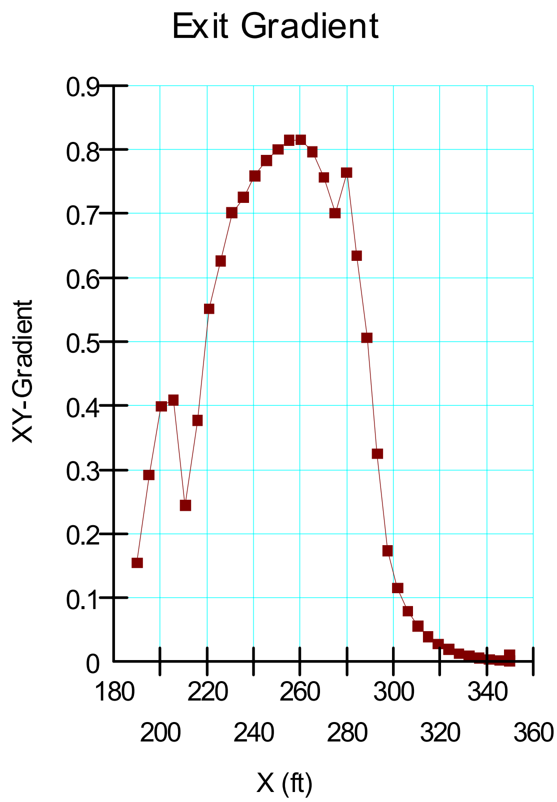
$W = (Same\ as\ Previous) = \ 1875\ psf$

$$FS_{UPLIFT} = \frac{W}{P} = \frac{1875}{1997} = 0.94$$

Uplift Factor of Safety calculations for tunnel head 8890 for Alternatives 0-2.



Seepage analysis results for tunnel head 8885 for Alternatives 0-2.



Exit gradient along the seepage face for tunnel elevation 8885 from seepage analysis for Alternatives 0-2.

<u>Uplift Pressure (P)</u>

$H = 8885 - 8858 = 27\ ft$

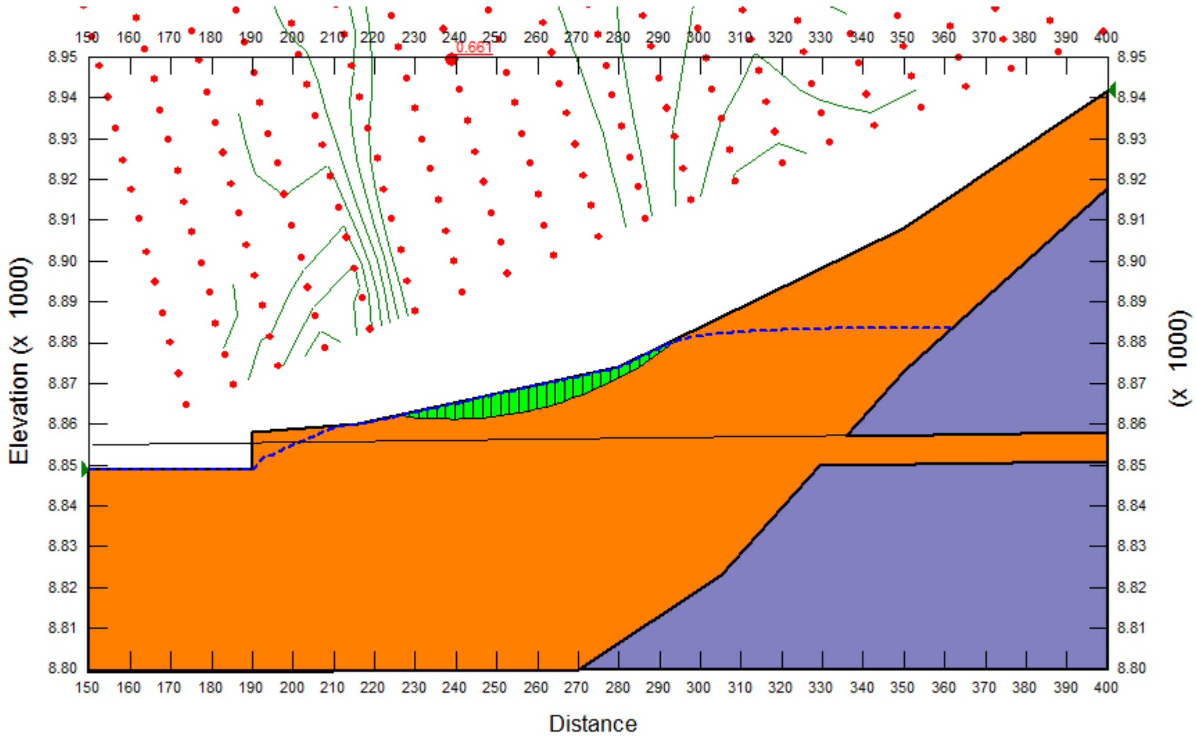
$P = 27\ ft\ x\ 62.4 = 1685\ psf$

<u>Weight Resisting Soil</u>

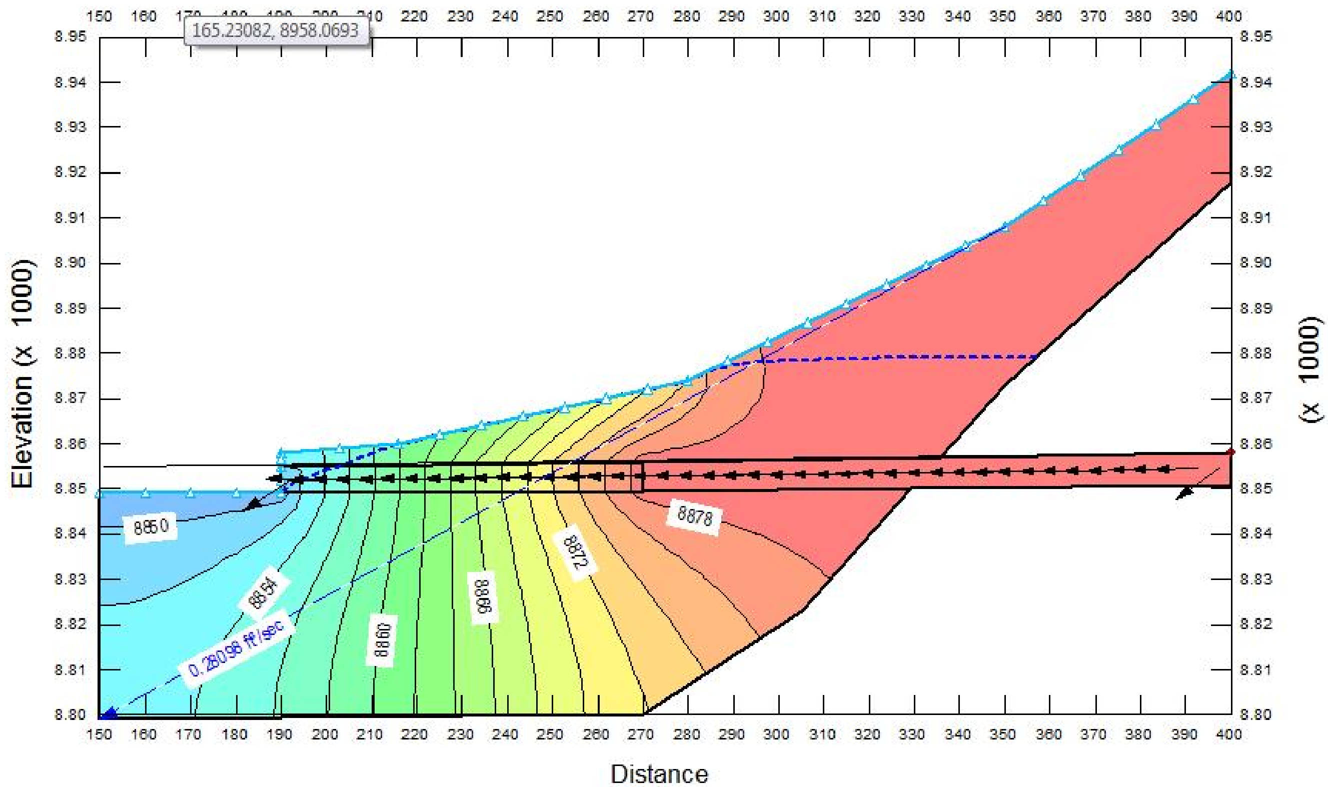
$W = 1875\ psf$

$FS_{UPLIFT} = \frac{W}{P} = 1.13$

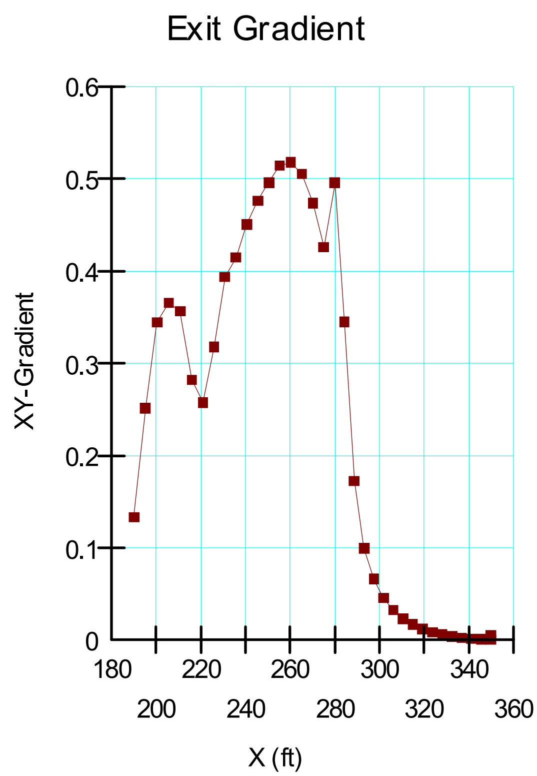
Uplift Factor of Safety calculations for tunnel head 8885 for Alternatives 0-2.



Results from steady seepage slope stability analysis for tunnel elevation 8885 for Alternatives 0-2.



Seepage analysis results for tunnel head 8880 for Alternatives 0-2.



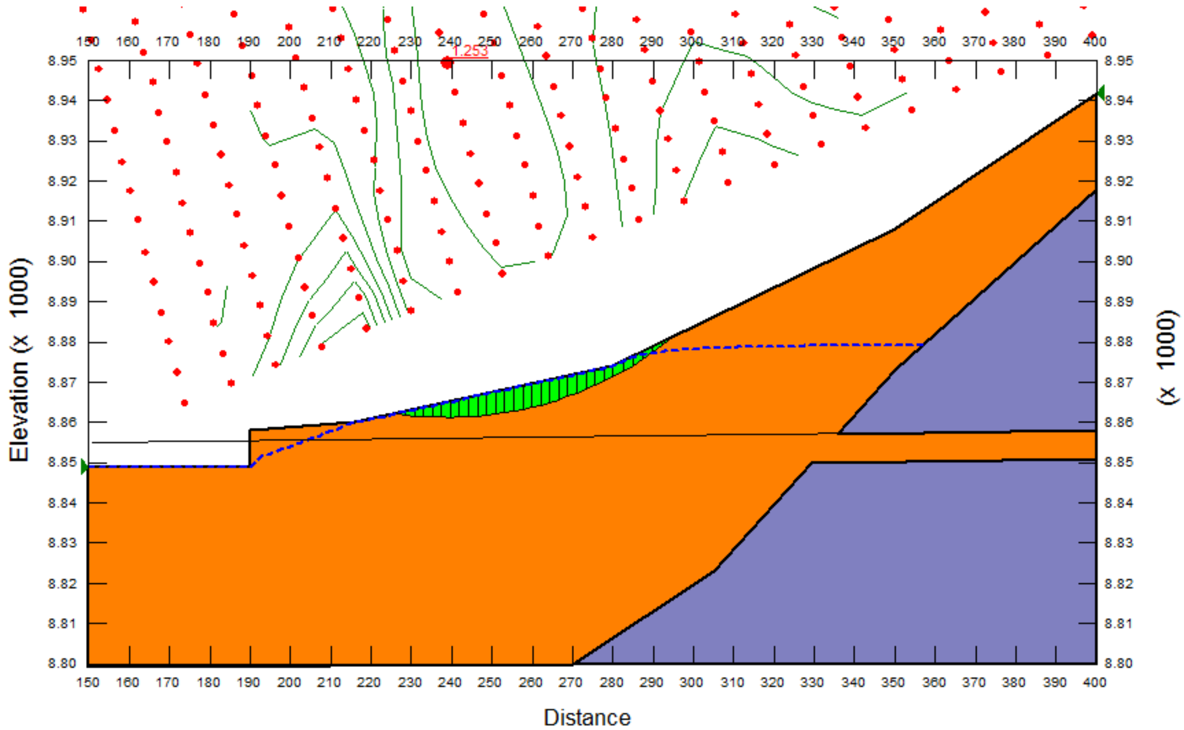
Exit gradient along the seepage face for tunnel elevation 8880 from seepage analysis for Alternatives 0-2**.**

Uplift Pressure (P)

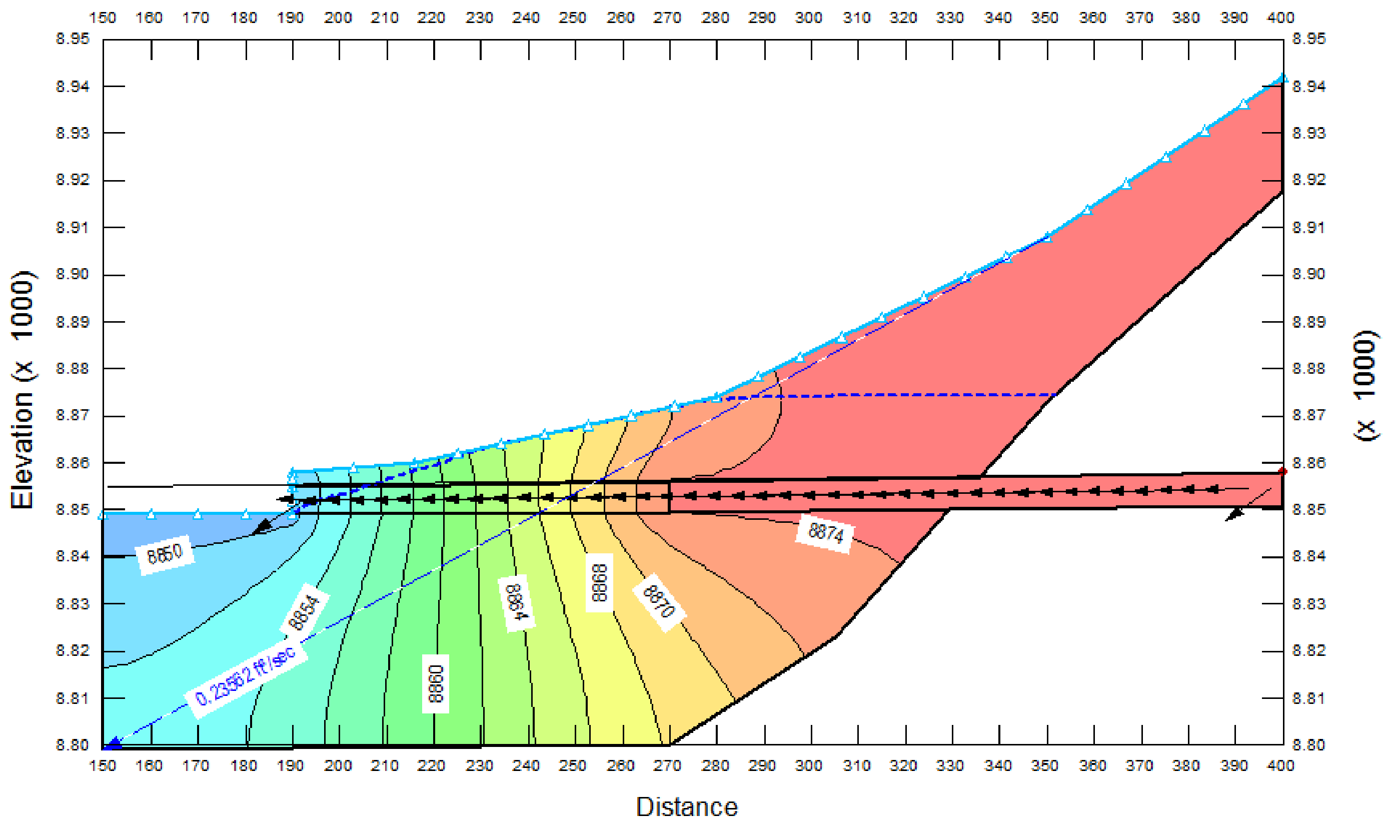
$H = 8880 - 8858 = 22\ ft$
$P = 22\ ft\ x\ 62.4 = 1373\ psf$

Weight Resisting Soil
$W = 1875\ psf$
$FS_{UPLIFT} = \frac{W}{P} = 1.37$

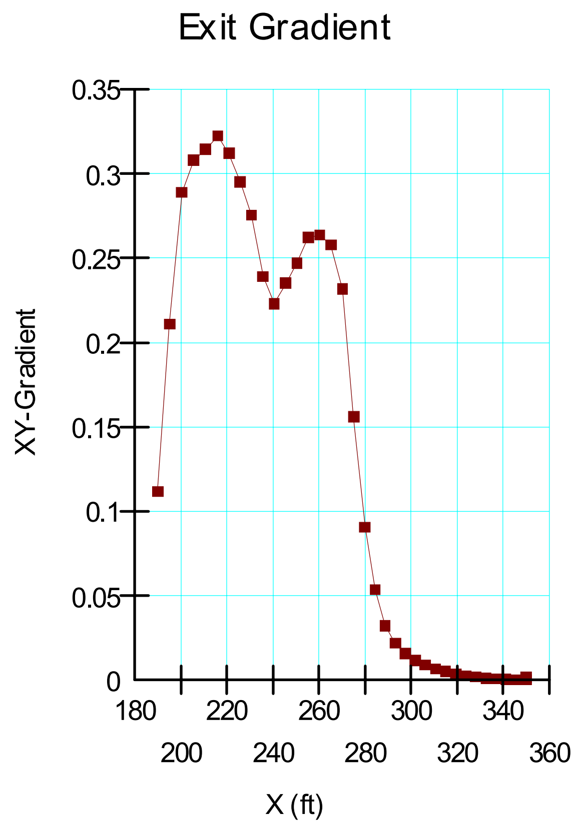
Uplift Factor of Safety calculations for tunnel head 8880 for Alternatives 0-2.



Results from steady seepage slope stability analysis for tunnel elevation 8880 for Alternatives 0-2.



Seepage analysis results for tunnel head 8875 for Alternatives 0-2.



Exit gradient along the seepage face for tunnel elevation 8875 from seepage analysis for Alternatives 0-2.

Uplift Pressure (P)

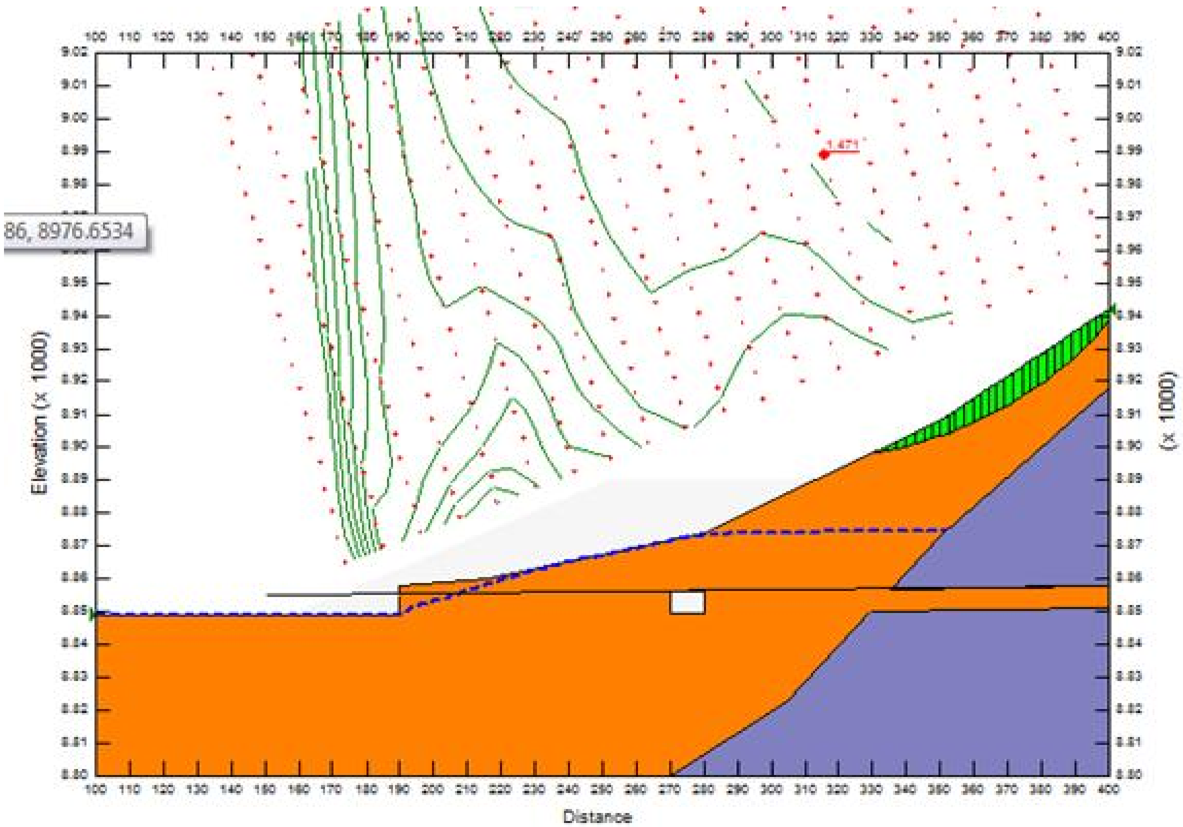
$H = 8875 - 8858 = 17\ ft$
$P = 17\ ft\ x\ 62.4 = 1060.8\ psf$

Weight Resisting Soil

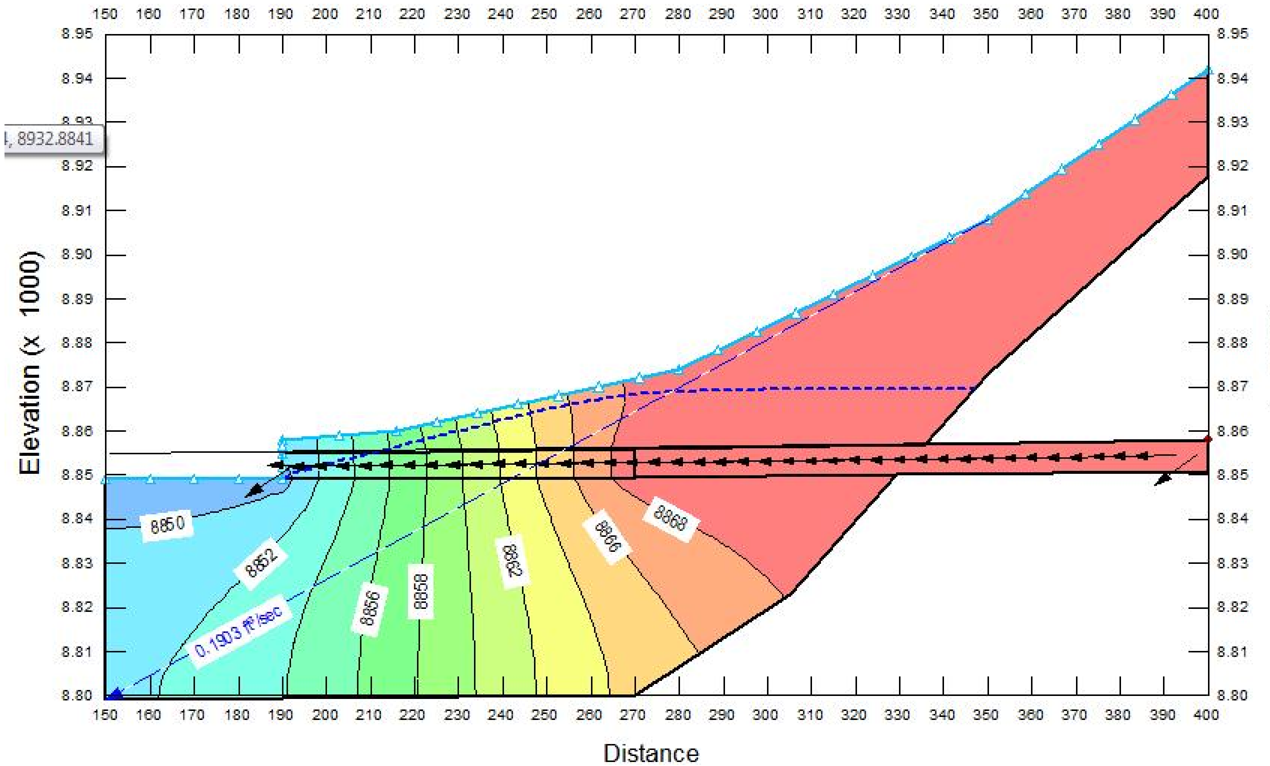
$W = 1875\ psf$

$FS_{UPLIFT} = \frac{W}{P} = 1.77$

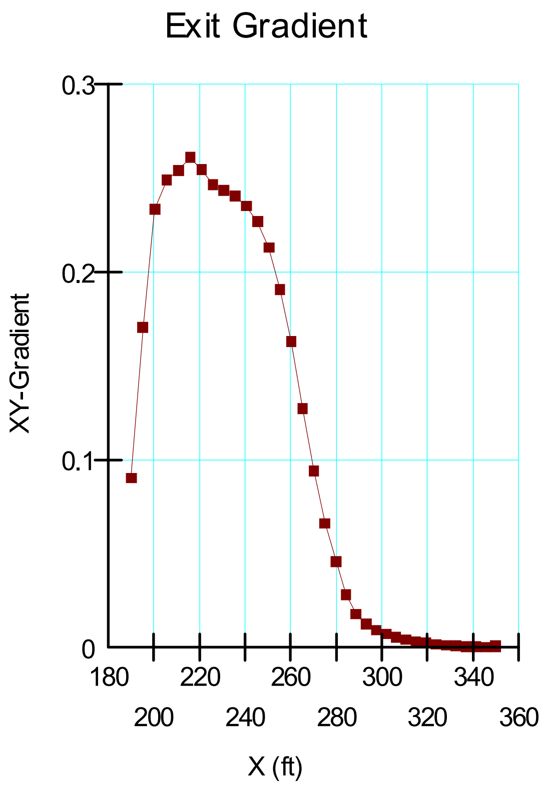
Uplift Factor of Safety calculations for tunnel head 8880 for Alternatives 0-2.



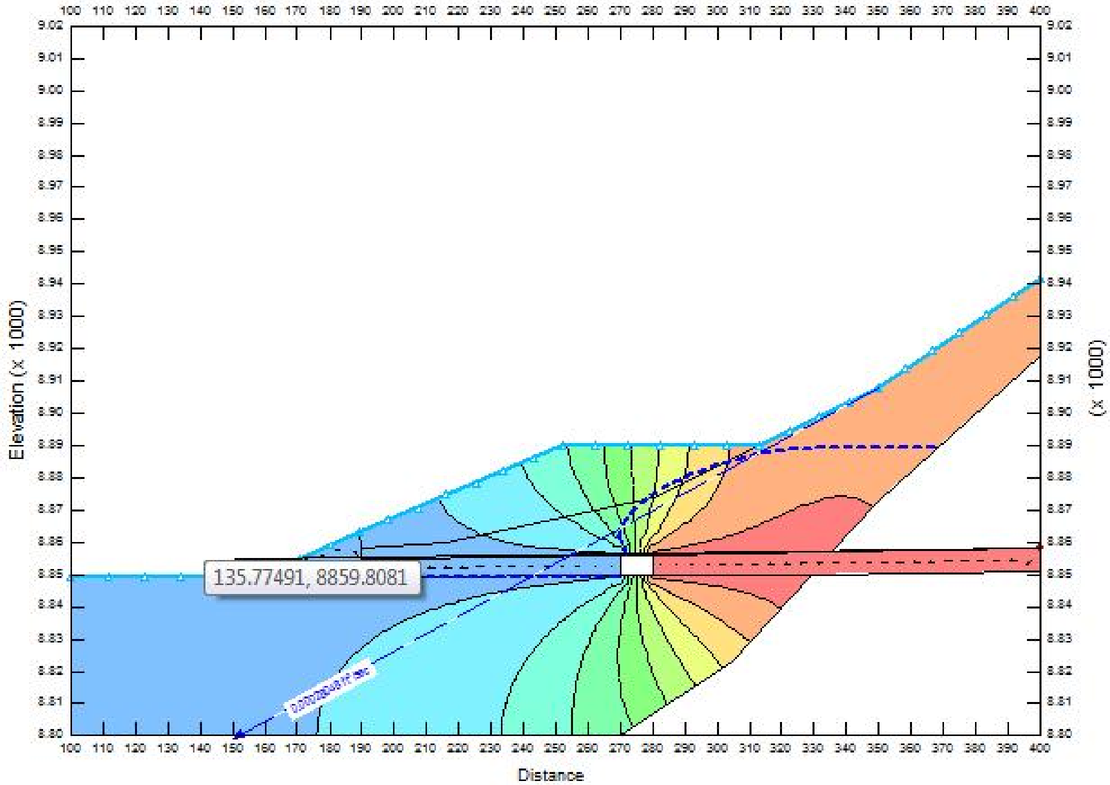
: Results from steady seepage slope stability analysis for tunnel elevation 8875 for Alternatives 0-2.



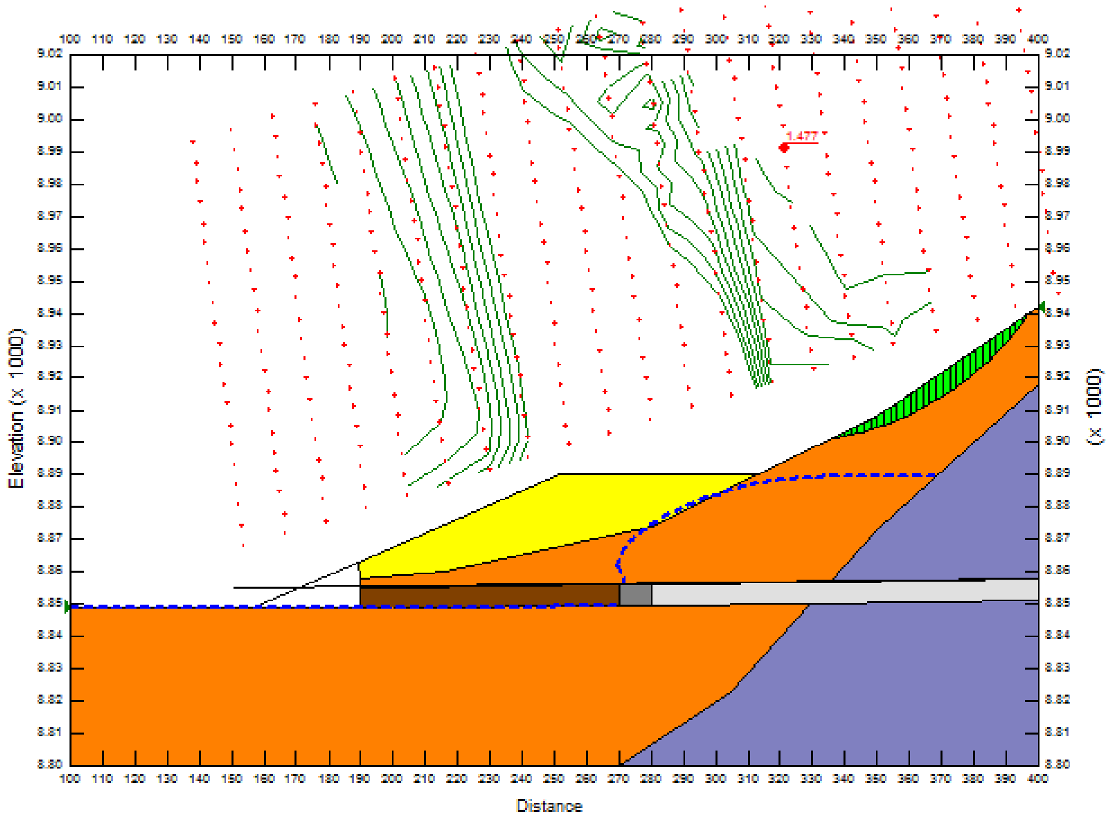
Seepage analysis results for tunnel head 8870 for Alternatives 0-2.



Exit gradient along the seepage face for tunnel elevation 8870 from seepage analysis for Alternatives 0-2.

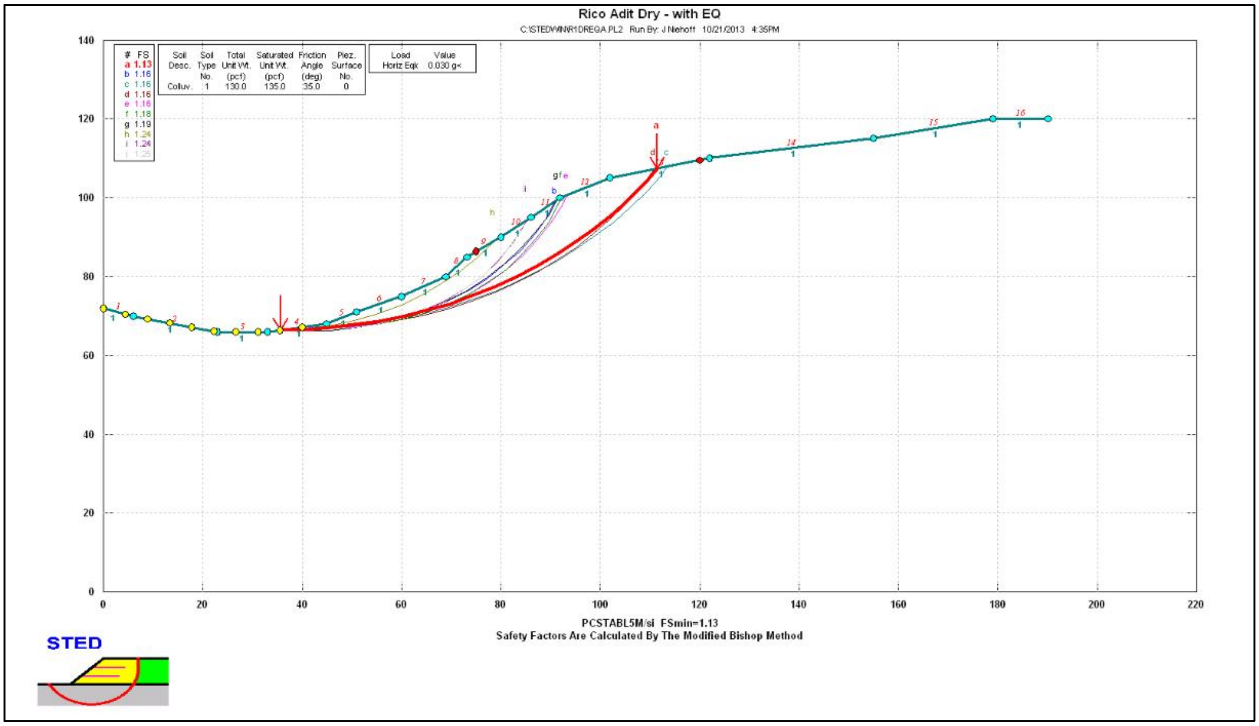


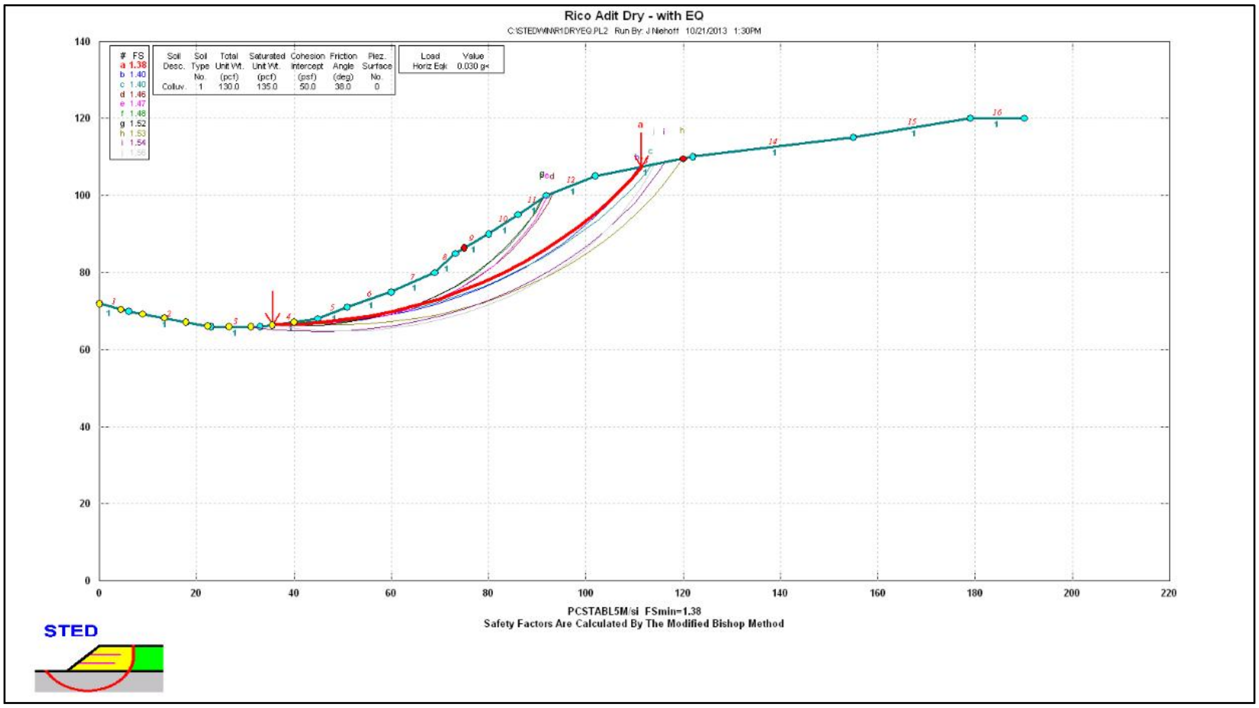
Seepage analysis results for tunnel head 8892 for Alternatives 3; no exit gradient.



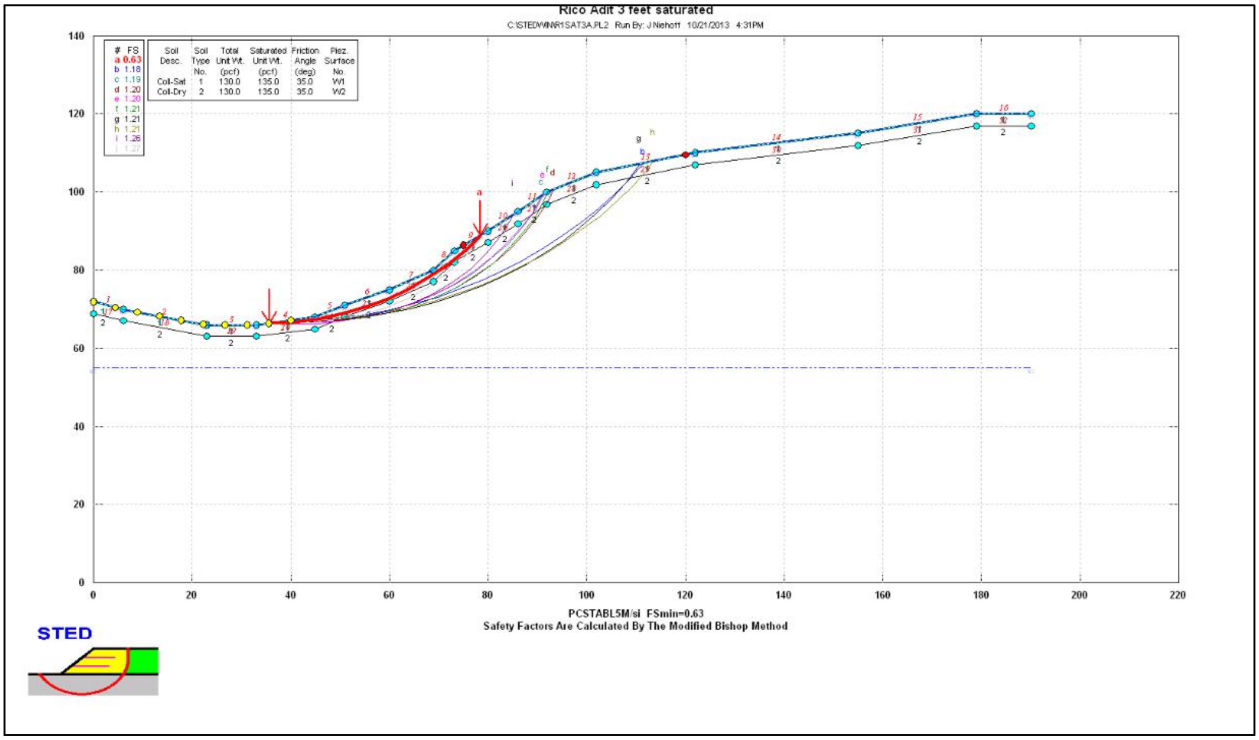
Results from steady seepage slope stability analysis for tunnel elevation 8892 for Alternative 3.

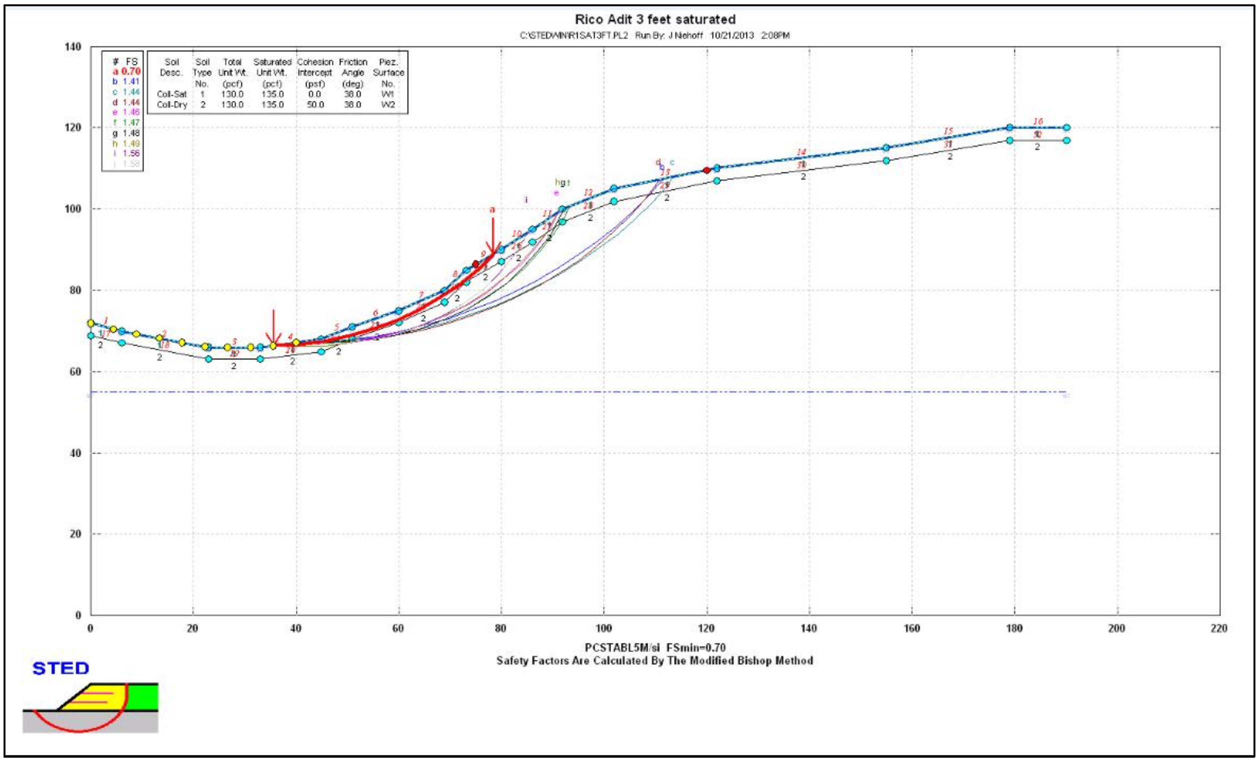
Terrain Trap and Adjacent Slopes - Stability


Rico Adit Dry - with EQ
C:\STED\WIN\R1DREQA.PL2 Run By: J Niehoff 10/21/2013 4:35PM
FS
a 1.13
b 1.16
c 1.16
d 1.16
e 1.16
f 1.18
g 1.19
h 1.24
i 1.24
j 1.25
Soil Desc. | Soil Type No. | Total Unit Wt. (pcf) | Saturated Unit Wt. (pcf) | Friction Angle (deg) | Piez. Surface No.
Colluv. | 1 | 130.0 | 135.0 | 35.0 | 0
Load | Value
Horiz Eqk | 0.030 g<
PCSTABL5M/si FSmin=1.13
Safety Factors Are Calculated By The Modified Bishop Method
STED

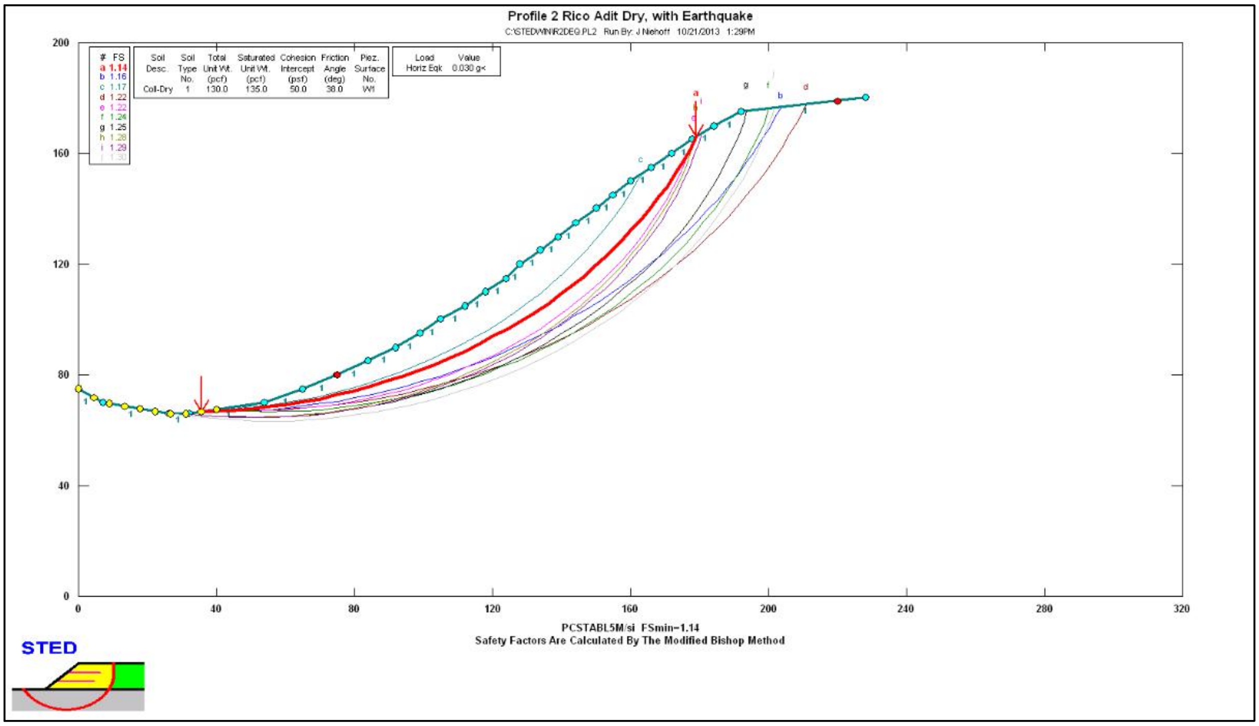


Rico Adit Dry - with EQ
C:\STED\WNR1DRYEQ.PL2 Run By: J Niehoff 10/21/2013 1:30PM
FS
a 1.38
b 1.40
c 1.40
d 1.46
e 1.47
f 1.48
g 1.52
h 1.53
i 1.54
j 1.56
Soil Desc. | Soil Type No. | Total Unit Wt. (pcf) | Saturated Unit Wt. (pcf) | Cohesion Intercept (psf) | Friction Angle (deg) | Piez. Surface No.
Colluv. | 1 | 130.0 | 135.0 | 50.0 | 38.0 | 0
Load | Value
Horiz Eqk | 0.030 g<
PCSTABL5M/si FSmin=1.38
Safety Factors Are Calculated By The Modified Bishop Method
STED

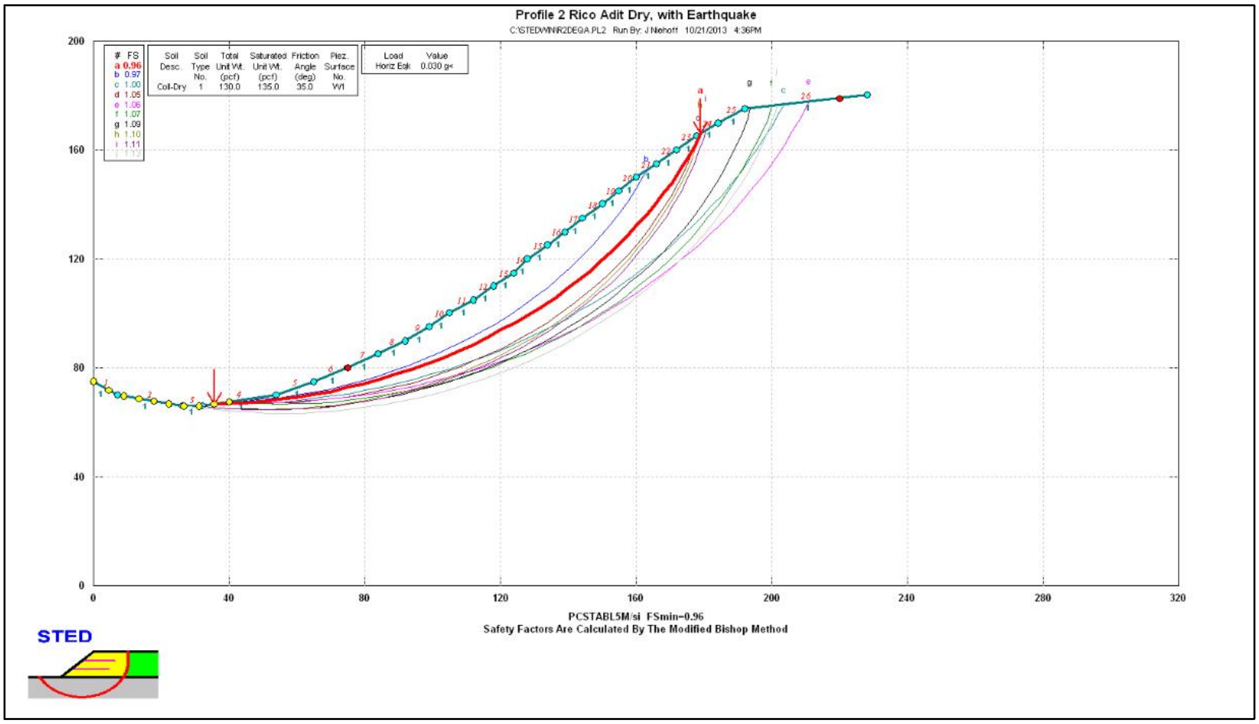

Rico Adit 3 feet saturated
C:\STED\WNR1SAT3A.PL2 Run By: J Niehoff 10/21/2013 4:31PM
FS
a 0.63
b 1.18
c 1.19
d 1.20
e 1.20
f 1.21
g 1.21
h 1.21
i 1.26
j 1.27
Soil Desc. | Soil Type No. | Total Unit Wt. (pcf) | Saturated Unit Wt. (pcf) | Friction Angle (deg) | Piez. Surface No.
Coll-Sat 1 130.0 135.0 35.0 W1
Coll-Dry 2 130.0 135.0 35.0 W2
140
120
100
80
60
40
20
0
0 20 40 60 80 100 120 140 160 180 200 220
PCSTABL5M/si FSmin=0.63
Safety Factors Are Calculated By The Modified Bishop Method
STED

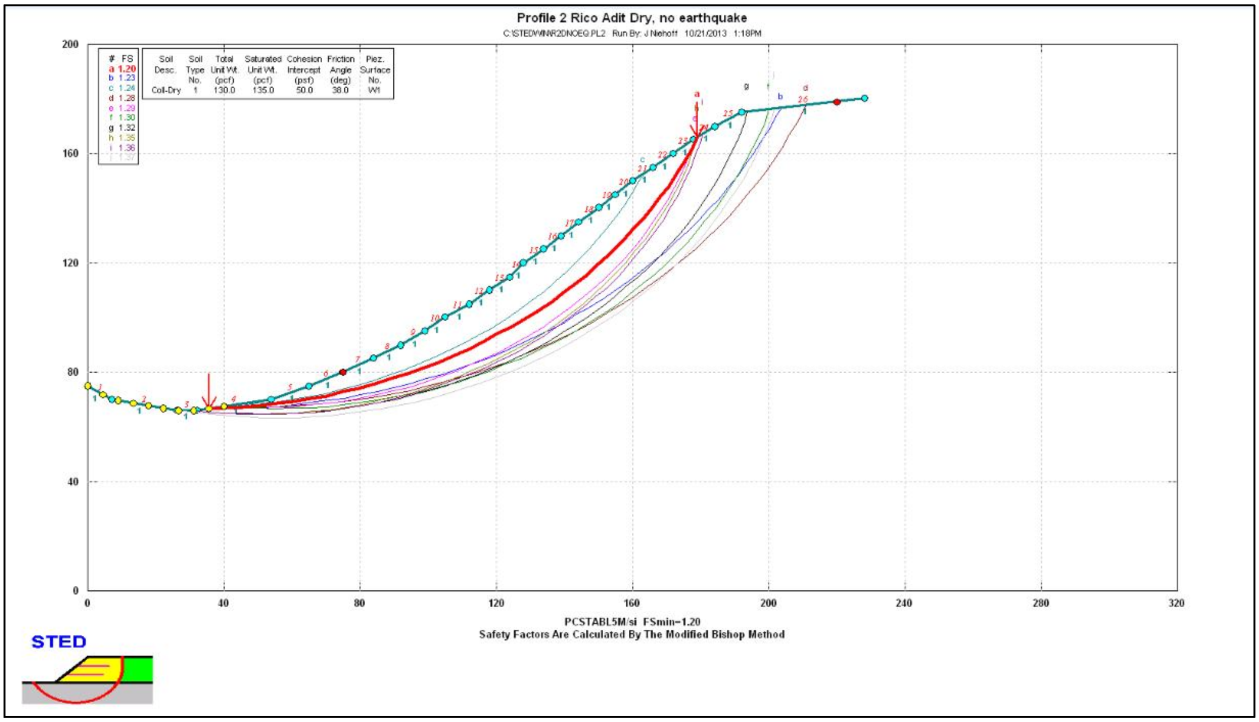


Rico Adit 3 feet saturated
C:\STEDWIN\R1SAT3FT.PL2 Run By: J Niehoff 10/21/2013 2:08PM
FS
a 0.70
b 1.41
c 1.44
d 1.44
e 1.46
f 1.47
g 1.48
h 1.49
i 1.56
j 1.58
Soil Desc. | Soil Type No. | Total Unit Wt. (pcf) | Saturated Unit Wt. (pcf) | Cohesion Intercept (psf) | Friction Angle (deg) | Piez. Surface No.
Coll-Sat 1 130.0 135.0 0.0 38.0 W1
Coll-Dry 2 130.0 135.0 50.0 38.0 W2
PCSTABL5M/si FSmin=0.70
Safety Factors Are Calculated By The Modified Bishop Method
STED

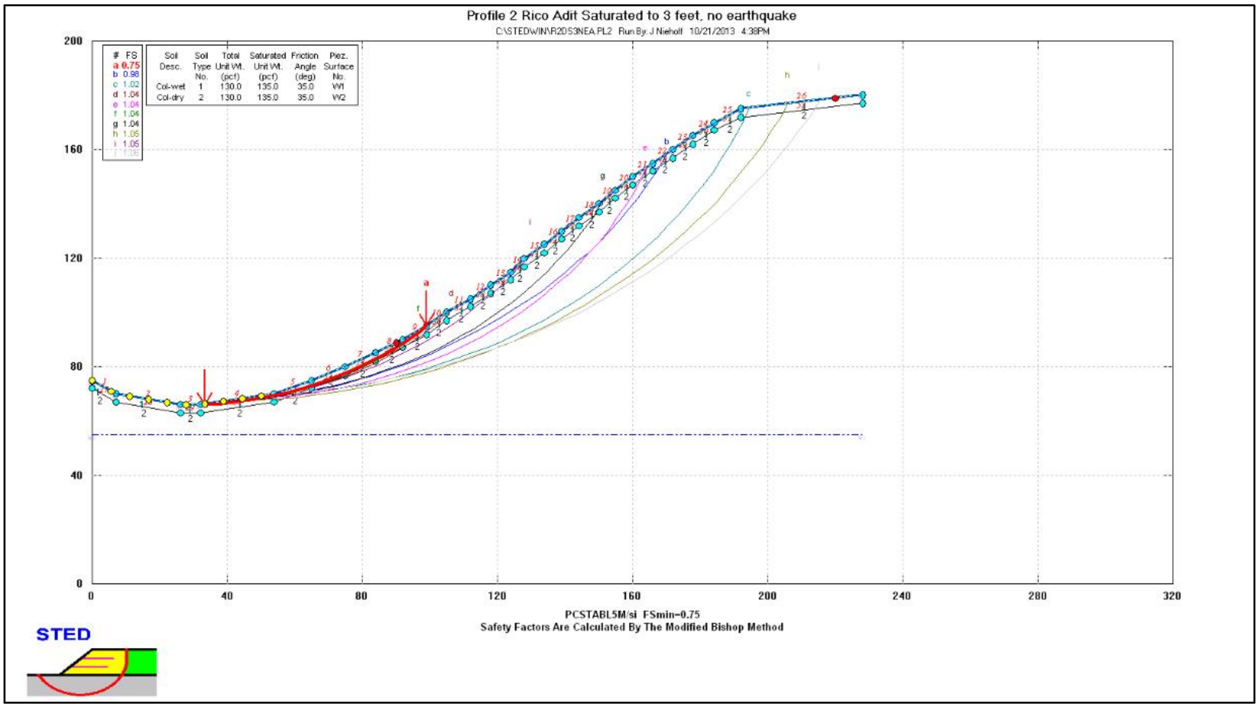


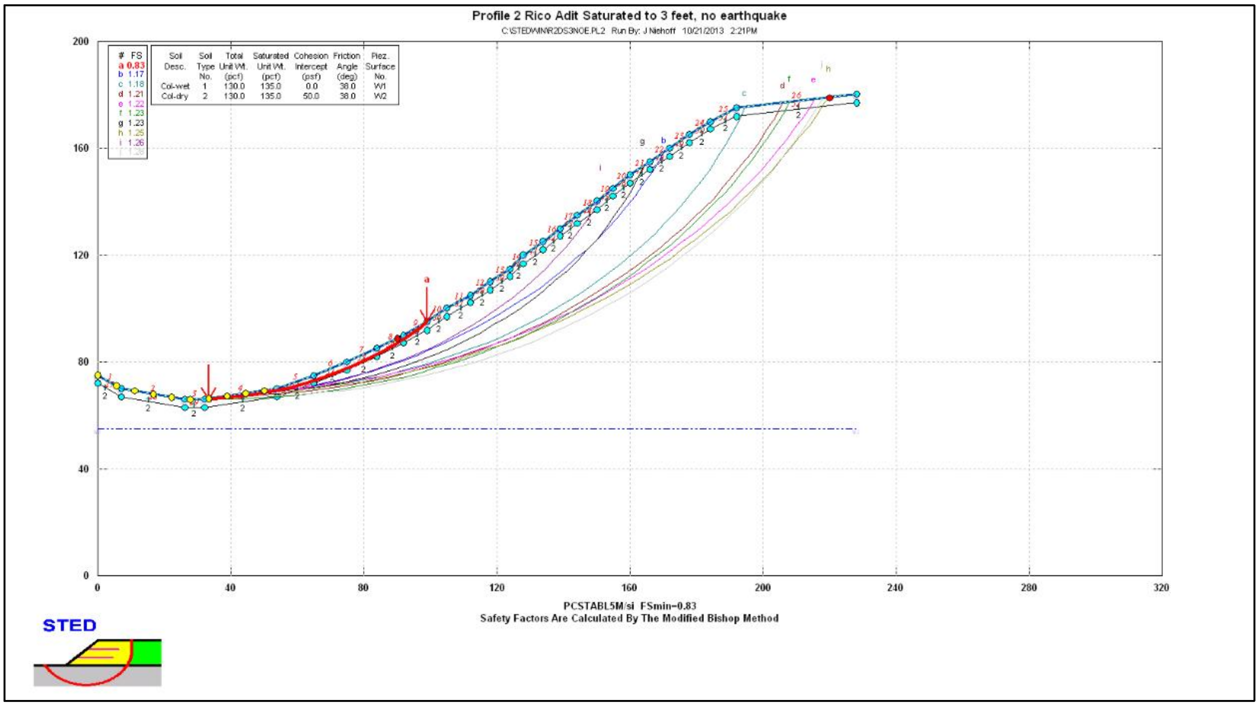
Profile 2 Rico Adit Dry, with Earthquake
C:\STEDWIN\R2DEQ.PL2 Run By: J Niehoff 10/21/2013 1:29PM
FS
a 1.14
b 1.16
c 1.17
d 1.22
e 1.22
f 1.24
g 1.25
h 1.28
i 1.29
j 1.30
Soil Desc. | Soil Type No. | Total Unit Wt. (pcf) | Saturated Unit Wt. (pcf) | Cohesion Intercept (psf) | Friction Angle (deg) | Piez. Surface No.
Coll-Dry | 1 | 130.0 | 135.0 | 50.0 | 38.0 | W1
Load | Value
Horiz Eqk | 0.030 g<
200
160
120
80
40
0
0
40
80
120
160
200
240
280
320
PCSTABL5M/si FSmin=1.14
Safety Factors Are Calculated By The Modified Bishop Method
STED


Profile 2 Rico Adit Dry, with Earthquake
C:\STEDWIN\R2DEGA.PL2 Run By: J Niehoff 10/21/2013 4:36PM
FS
a 0.96
b 0.97
c 1.00
d 1.05
e 1.06
f 1.07
g 1.09
h 1.10
i 1.11
Soil Desc. | Soil Type No. | Total Unit Wt. (pcf) | Saturated Unit Wt. (pcf) | Friction Angle (deg) | Piez. Surface No.
Coll-Dry | 1 | 130.0 | 135.0 | 35.0 | W1
Load | Value
Horiz Eqk | 0.030 g<
200
160
120
80
40
0
0
40
80
120
160
200
240
280
320
PCSTABL5M/si FSmin=0.96
Safety Factors Are Calculated By The Modified Bishop Method
STED

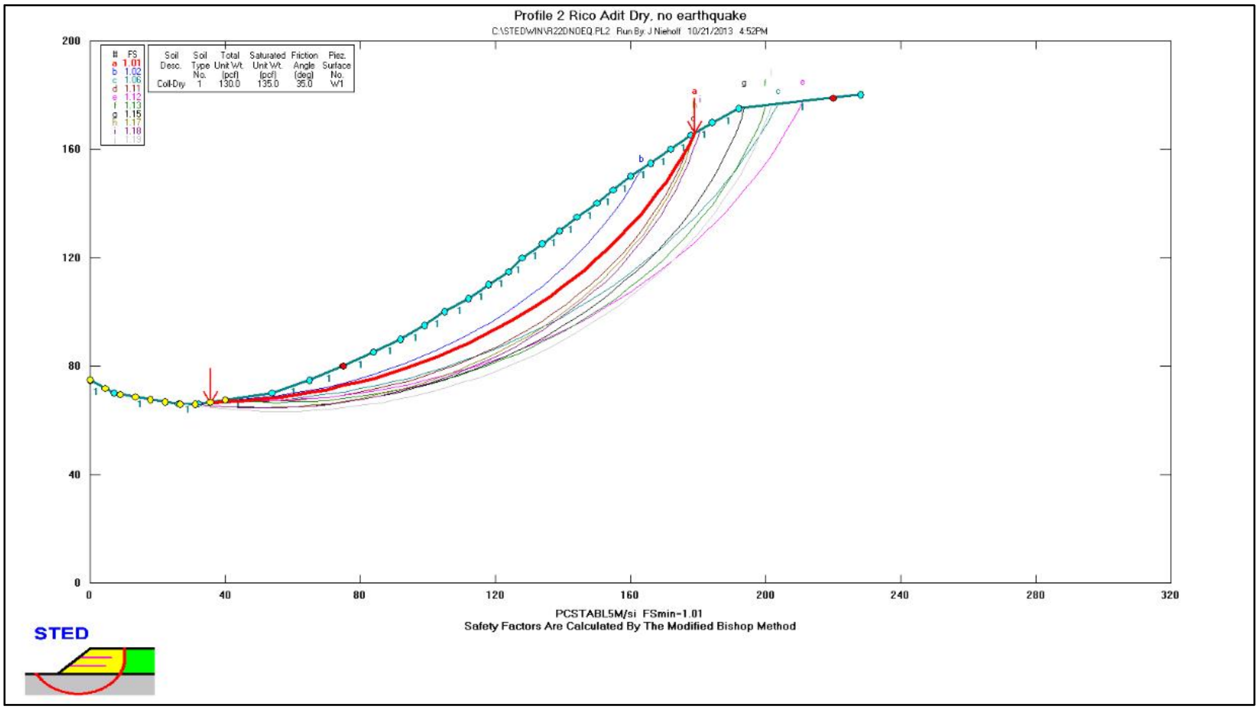


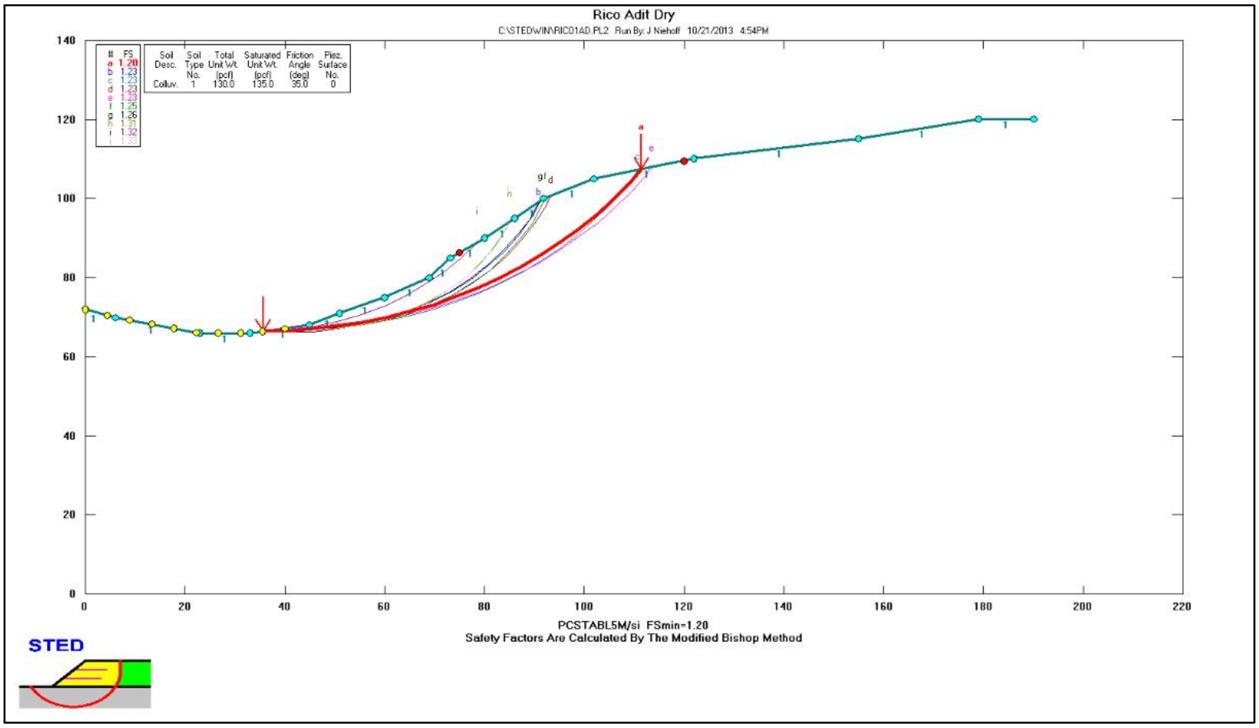
Profile 2 Rico Adit Dry, no earthquake
C:\STEDWIN\R2DNOEQ.PL2 Run By: J Niehoff 10/21/2013 1:18PM
FS
a 1.20
b 1.23
c 1.24
d 1.28
e 1.29
f 1.30
g 1.32
h 1.35
i 1.36
j 1.37
Soil Desc. | Soil Type No. | Total Unit Wt. (pcf) | Saturated Unit Wt. (pcf) | Cohesion Intercept (psf) | Friction Angle (deg) | Piez. Surface No.
Coll-Dry | 1 | 130.0 | 135.0 | 50.0 | 38.0 | W1
200
160
120
80
40
0
0
40
80
120
160
200
240
280
320
PCSTABL5M/si FSmin=1.20
Safety Factors Are Calculated By The Modified Bishop Method
STED


Profile 2 Rico Adit Saturated to 3 feet, no earthquake
C:\STEDWIN\R2DS3NEA.PL2 Run By: J Niehoff 10/21/2013 4:38PM
FS
a 0.75
b 0.98
c 1.02
d 1.04
e 1.04
f 1.04
g 1.04
h 1.05
i 1.05
Soil Desc. | Soil Type No. | Total Unit Wt. (pcf) | Saturated Unit Wt. (pcf) | Friction Angle (deg) | Piez. Surface No.
Col-wet 1 130.0 135.0 35.0 W1
Col-dry 2 130.0 135.0 35.0 W2
PCSTABL5M/si FSmin=0.75
Safety Factors Are Calculated By The Modified Bishop Method
STED

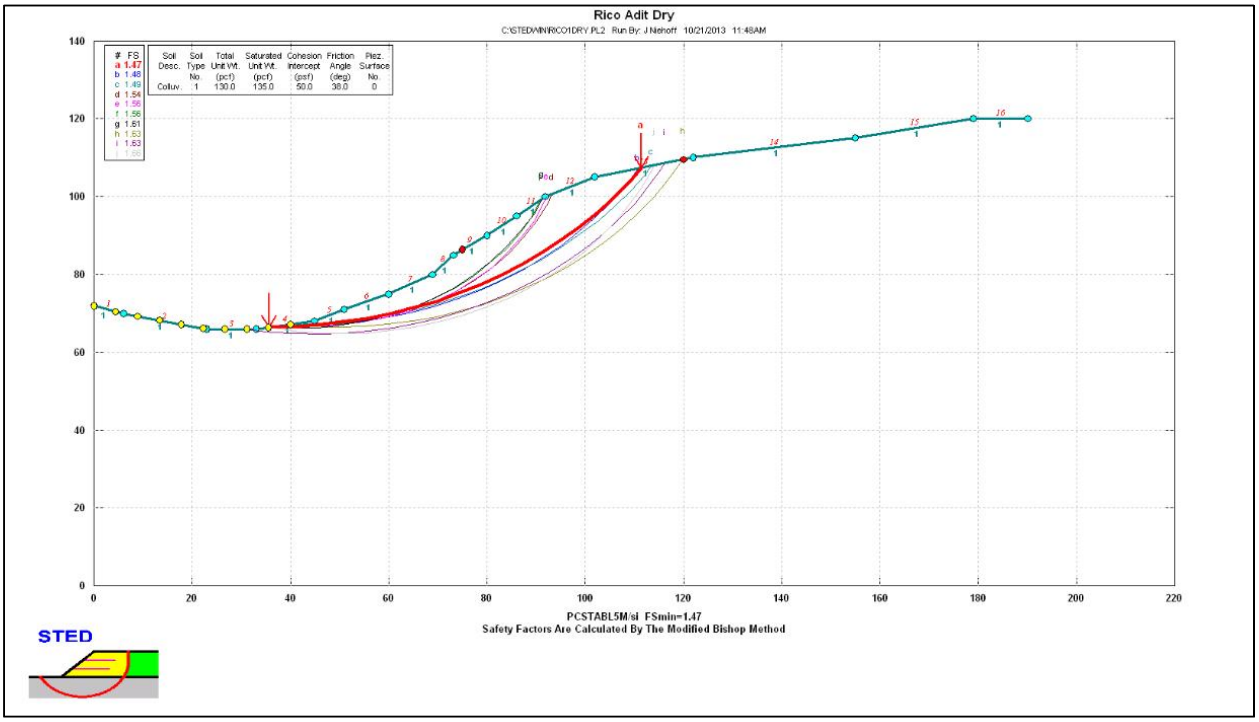


Profile 2 Rico Adit Saturated to 3 feet, no earthquake
C:\STEDWIN\R2DS3NOE.PL2 Run By: J Niehoff 10/21/2013 2:21PM
FS
a 0.83
b 1.17
c 1.18
d 1.21
e 1.22
f 1.23
g 1.23
h 1.25
i 1.26
j 1.26
Soil Desc. | Soil Type No. | Total Unit Wt. (pcf) | Saturated Unit Wt. (pcf) | Cohesion Intercept (psf) | Friction Angle (deg) | Piez. Surface No.
Col-wet 1 130.0 135.0 0.0 38.0 W1
Col-dry 2 130.0 135.0 50.0 38.0 W2
200
160
120
80
40
0
0
40
80
120
160
200
240
280
320
PCSTABL5M/si FSmin=0.83
Safety Factors Are Calculated By The Modified Bishop Method
STED

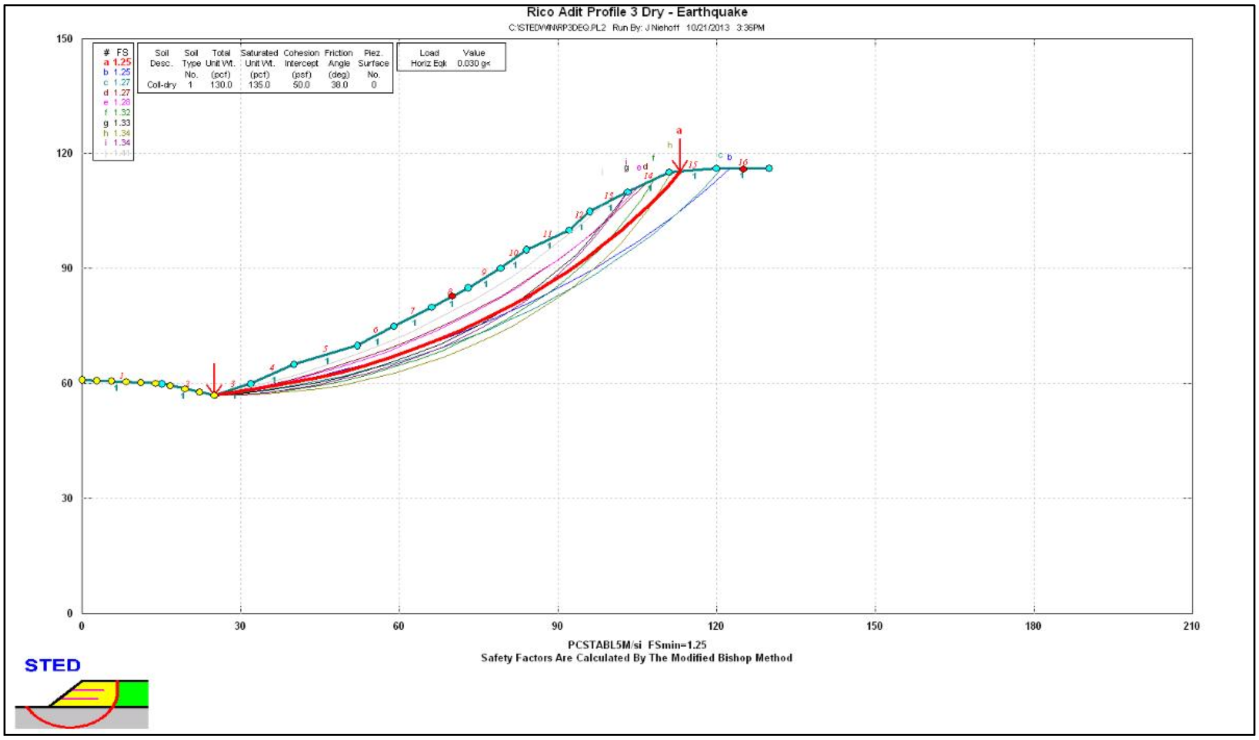

Profile 2 Rico Adit Dry, no earthquake
C:\STEDWIN\R22DNOEQ.PL2 Run By: J Nieholf 10/21/2013 4:52PM
FS
a 1.01
b 1.02
c 1.06
d 1.11
e 1.12
f 1.13
g 1.15
h 1.17
i 1.18
j 1.19
Soil Desc. | Soil Type No. | Total Unit Wt. (pcf) | Saturated Unit Wt. (pcf) | Friction Angle (deg) | Piez. Surface No.
Coll-Dry | 1 | 130.0 | 135.0 | 35.0 | W1
200
160
120
80
40
0
0
40
80
120
160
200
240
280
320
PCSTABL5M/si FSmin=1.01
Safety Factors Are Calculated By The Modified Bishop Method
STED

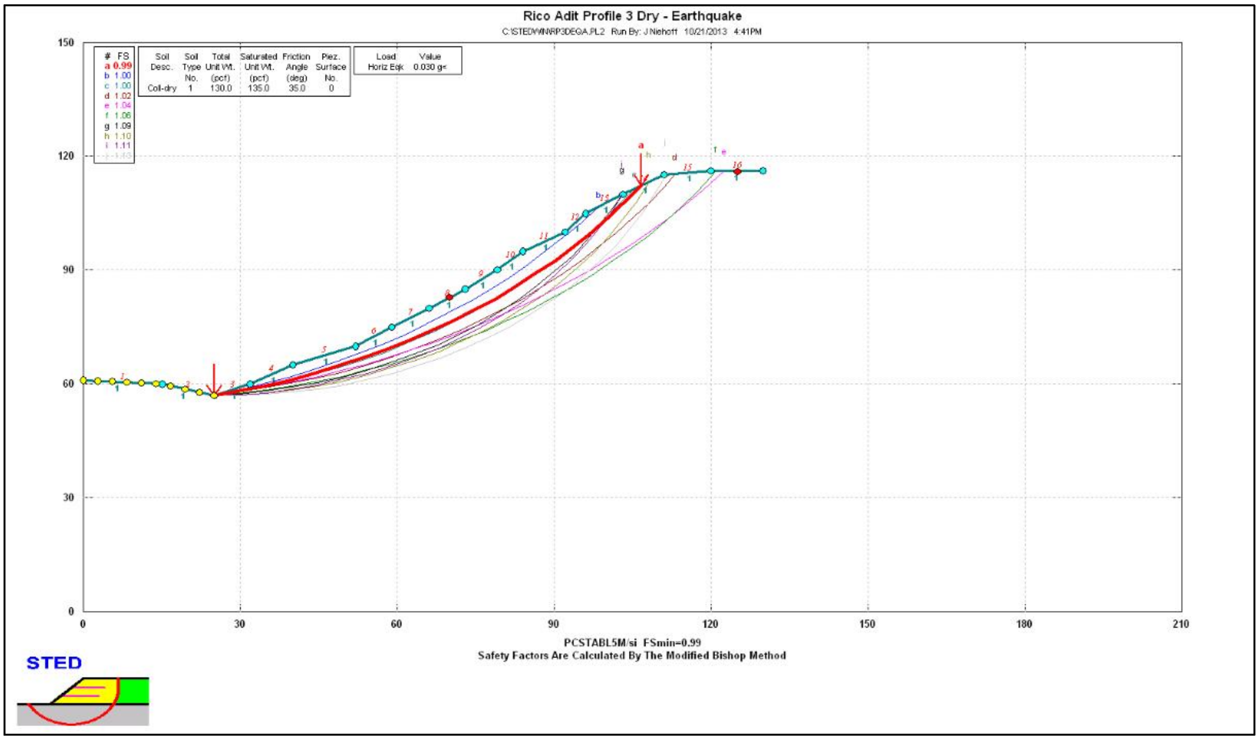


Rico Adit Dry
C:\STEDWIN\RICO1AD.PL2 Run By: J Niehoff 10/21/2013 4:54PM
FS
a 1.20
b 1.23
c 1.23
d 1.23
e 1.23
f 1.25
g 1.26
h 1.31
i 1.32
Soil Desc. | Soil Type No. | Total Unit Wt. (pcf) | Saturated Unit Wt. (pcf) | Friction Angle (deg) | Piez. Surface No.
Colluv. | 1 | 130.0 | 135.0 | 35.0 | 0
140
120
100
80
60
40
20
0
0 20 40 60 80 100 120 140 160 180 200 220
PCSTABL5M/si FSmin=1.20
Safety Factors Are Calculated By The Modified Bishop Method
STED

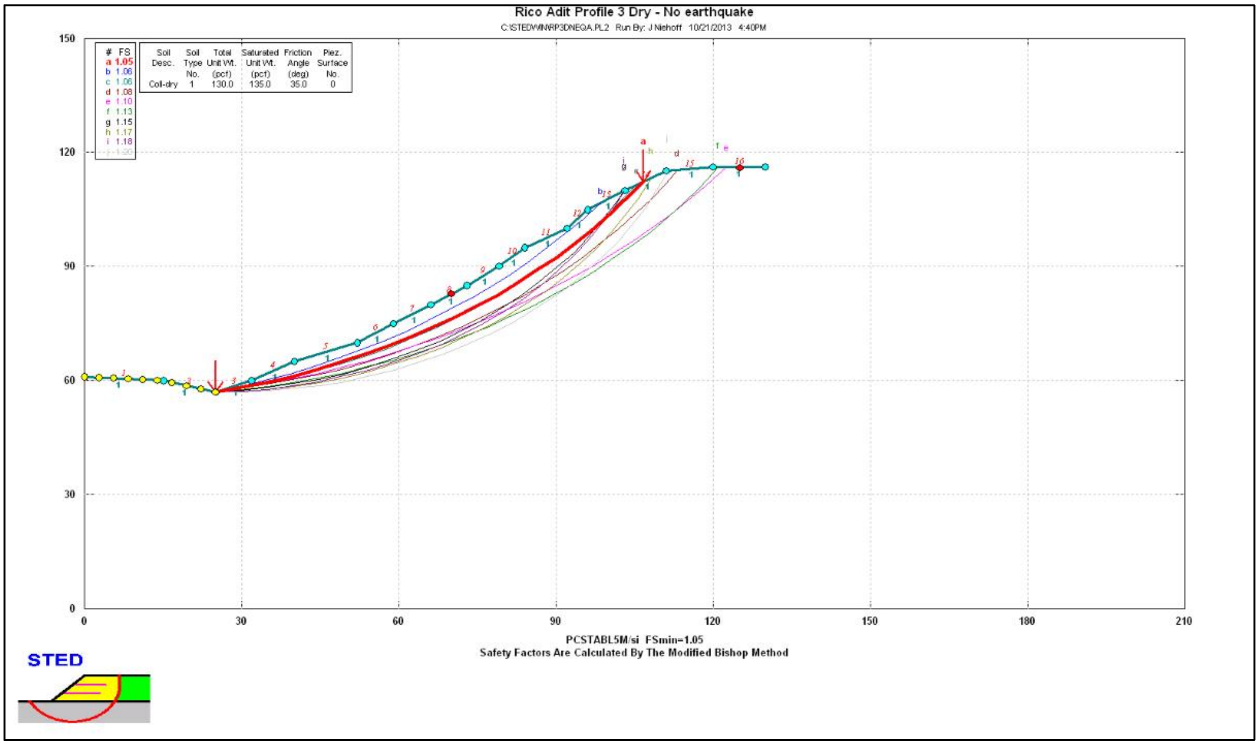


Rico Adit Dry
C:\STEDWIN\RICO1DRY.PL2 Run By: J Niehoff 10/21/2013 11:48AM
FS
a 1.47
b 1.48
c 1.49
d 1.54
e 1.56
f 1.56
g 1.61
h 1.63
i 1.63
j 1.66
Soil Desc. | Soil Type No. | Total Unit Wt. (pcf) | Saturated Unit Wt. (pcf) | Cohesion Intercept (psf) | Friction Angle (deg) | Piez. Surface No.
Colluv. | 1 | 130.0 | 135.0 | 50.0 | 38.0 | 0
PCSTABL5M/si FSmin=1.47
Safety Factors Are Calculated By The Modified Bishop Method
STED

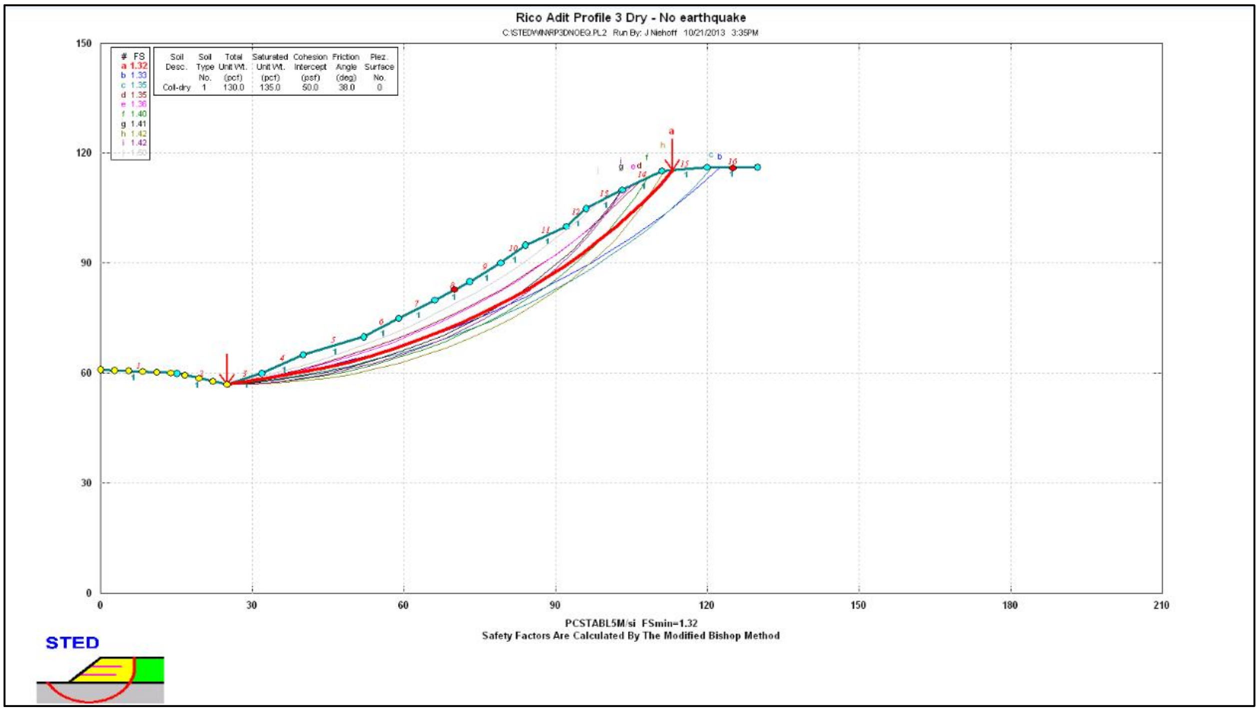

Rico Adit Profile 3 Dry - Earthquake
C:\STEDWIN\RP3DEQ.PL2 Run By: J Niehoff 10/21/2013 3:36PM
FS
a 1.25
b 1.25
c 1.27
d 1.27
e 1.28
f 1.32
g 1.33
h 1.34
i 1.34
Soil Desc. | Soil Type No. | Total Unit Wt. (pcf) | Saturated Unit Wt. (pcf) | Cohesion Intercept (psf) | Friction Angle (deg) | Piez. Surface No.
Coll-dry | 1 | 130.0 | 135.0 | 50.0 | 38.0 | 0
Load | Value
Horiz Eqk | 0.030 g<
PCSTABL5M/si FSmin=1.25
Safety Factors Are Calculated By The Modified Bishop Method
STED

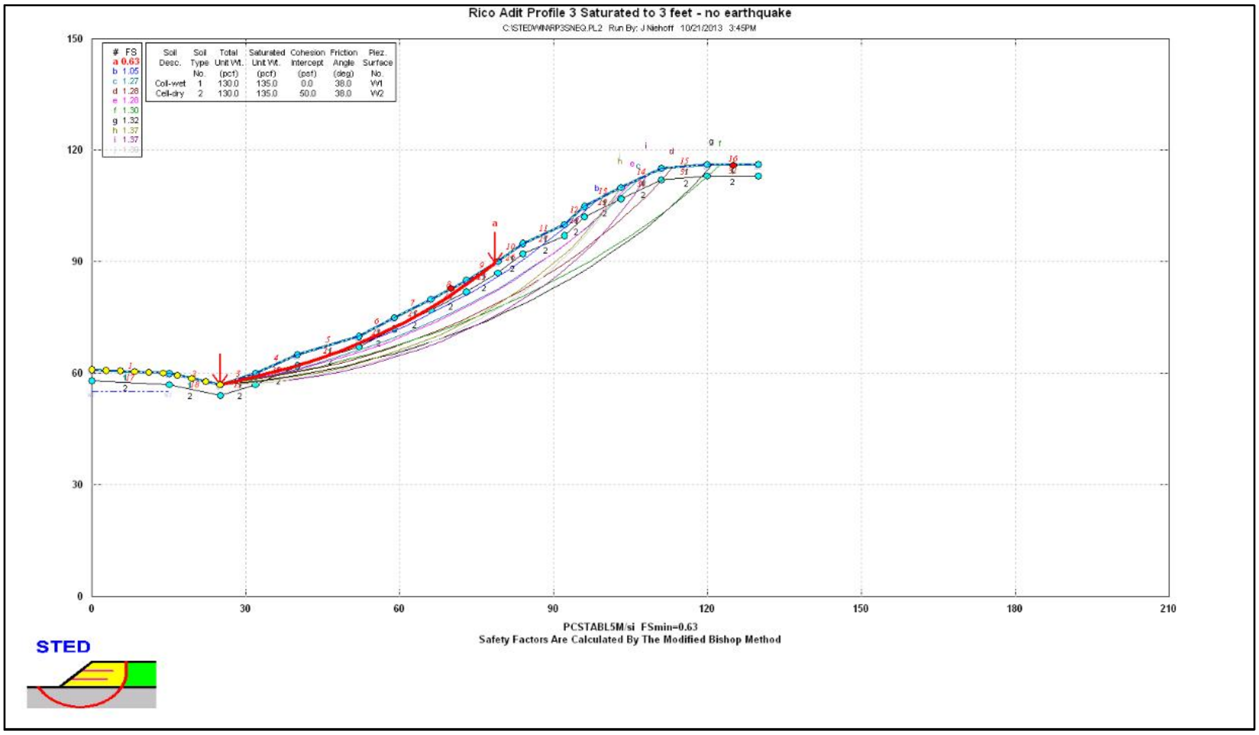


Rico Adit Profile 3 Dry - Earthquake
C:\STEDWIN\RP3DEQA.PL2 Run By: J Niehoff 10/21/2013 4:41PM
FS
a 0.99
b 1.00
c 1.00
d 1.02
e 1.04
f 1.06
g 1.09
h 1.10
i 1.11
Soil Desc. | Soil Type No. | Total Unit Wt. (pcf) | Saturated Unit Wt. (pcf) | Friction Angle (deg) | Piez. Surface No.
Coll-dry | 1 | 130.0 | 135.0 | 35.0 | 0
Load | Value
Horiz Eqk | 0.030 g<
150
120
90
60
30
0
0
30
60
90
120
150
180
210
PCSTABL5M/si FSmin=0.99
Safety Factors Are Calculated By The Modified Bishop Method
STED

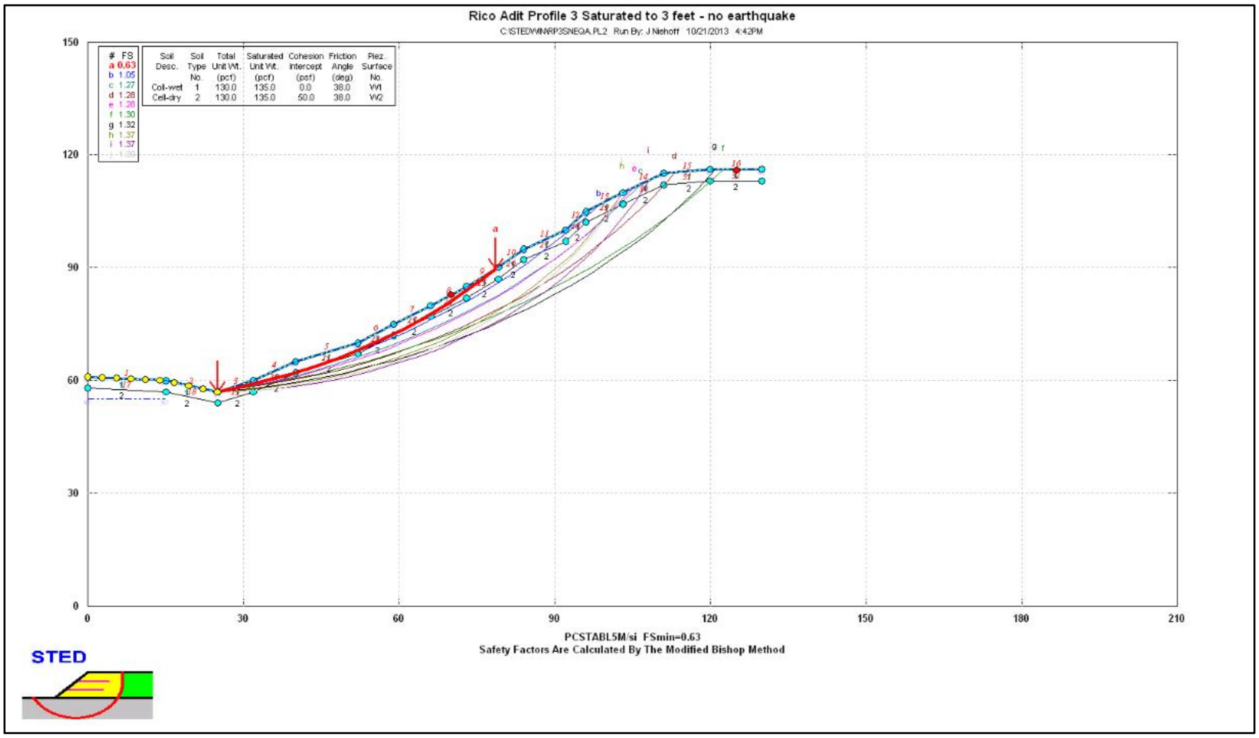


Rico Adit Profile 3 Dry - No earthquake
C:\STEDWIN\RP3DNEQA.PL2 Run By: J Niehoff 10/21/2013 4:40PM
FS
a 1.05
b 1.06
c 1.06
d 1.08
e 1.10
f 1.13
g 1.15
h 1.17
i 1.18
j 1.20
Soil Desc. | Soil Type No. | Total Unit Wt. (pcf) | Saturated Unit Wt. (pcf) | Friction Angle (deg) | Piez. Surface No.
Coll-dry 1 130.0 135.0 35.0 0
150
120
90
60
30
0
0
30
60
90
120
150
180
210
PCSTABL5M/si FSmin=1.05
Safety Factors Are Calculated By The Modified Bishop Method
STED


Rico Adit Profile 3 Dry - No earthquake
C:\STEDWIN\RP3DNOEQ.PL2 Run By: J Niehoff 10/21/2013 3:35PM
FS
a 1.32
b 1.33
c 1.35
d 1.35
e 1.36
f 1.40
g 1.41
h 1.42
i 1.42
j 1.50
Soil Desc. | Soil Type No. | Total Unit Wt. (pcf) | Saturated Unit Wt. (pcf) | Cohesion Intercept (psf) | Friction Angle (deg) | Piez. Surface No.
Coll-dry | 1 | 130.0 | 135.0 | 50.0 | 38.0 | 0
PCSTABL5M/si FSmin=1.32
Safety Factors Are Calculated By The Modified Bishop Method
STED



Rico Adit Profile 3 Saturated to 3 feet - no earthquake
C:\STEDWIN\RP3SNEQ.PL2 Run By: J Niehoff 10/21/2013 3:45PM
FS
a 0.63
b 1.05
c 1.27
d 1.28
e 1.28
f 1.30
g 1.32
h 1.37
i 1.37
Soil Desc. | Soil Type No. | Total Unit Wt. (pcf) | Saturated Unit Wt. (pcf) | Cohesion Intercept (psf) | Friction Angle (deg) | Piez. Surface No.
Coll-wet | 1 | 130.0 | 135.0 | 0.0 | 38.0 | W1
Cell-dry | 2 | 130.0 | 135.0 | 50.0 | 38.0 | W2
150
120
90
60
30
0
0
30
60
90
120
150
180
210
PCSTABL5M/si FSmin=0.63
Safety Factors Are Calculated By The Modified Bishop Method
STED



Rico Adit Profile 3 Saturated to 3 feet - no earthquake
C:\STEDWIN\RP3SNEQA.PL2 Run By: J Niehoff 10/21/2013 4:42PM
FS
a 0.63
b 1.05
c 1.27
d 1.28
e 1.28
f 1.30
g 1.32
h 1.37
i 1.37
Soil Desc. | Soil Type No. | Total Unit Wt. (pcf) | Saturated Unit Wt. (pcf) | Cohesion Intercept (psf) | Friction Angle (deg) | Piez. Surface No.
Coll-wet 1 130.0 135.0 0.0 38.0 W1
Cell-dry 2 130.0 135.0 50.0 38.0 W2
PCSTABL5M/si FSmin=0.63
Safety Factors Are Calculated By The Modified Bishop Method
STED